

**MINERALS MANAGEMENT SERVICE**

**HURRICANE ANDREW  
PLATFORM INSPECTION AND ANALYSIS**

**FINAL REPORT**

**PMB ENGINEERING INC.  
SAN FRANCISCO, CALIFORNIA**

**JUNE, 1994**

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## **EXECUTIVE SUMMARY**

### **Background**

Hurricane Andrew (Andrew) passed through a heavily populated region of offshore platforms during August, 1992. The Minerals Management Service subsequently contracted with PMB Engineering Inc. (PMB) to perform independent underwater inspections of several platforms that were located in some of the most severe portions of the storm. PMB also used computer analysis to see if the observed platform behavior (based upon the inspections) could be predicted analytically. Results of the Andrew Joint Industry Project (JIP) which evaluated the performance of 13 platforms in Andrew were used as a basis to help select the platforms, define the inspection program and provide an initial basis for the computer models (PMB, 1993).

### **Inspections**

Three platforms were selected and inspected during November 1993. The inspection work was subcontracted to Oceaneering. All of the platforms were owned by Chevron who was most cooperative and forthcoming for this project. The inspections included a check of all primary members for flooding, and Magnetic Particle Inspection (MPI) of members and joints that were either heavily loaded during Andrew or were found to be flooded. The inspections were of greater detail than typical visual-only inspections performed by operators in response to MMS NTL 92-07 immediately following Andrew.

A description of the platforms inspected and key results of the inspection are as follows:

**ST 130 A** - Installed in 1958, 140 ft water depth. Toppled in Andrew. The platform toppled in basically a single piece providing an opportunity to inspect the debris which were in good shape. This deck had been previously raised by the operator and therefore it is estimated that waves from Andrew did not hit the deck of the platform. The inspection revealed clear evidence of bending failure of the legs in the second bay above the seafloor. The leg-pile annulus was supposed to be fully grouted, however, there appeared to be no grout in the vicinity of the failures, although Chevron indicates that a previous inspection did find grout in this area. There were initial signs of joint failure at the X and K joints, but it appears that the platform toppled prior to complete failure of these joints. There were no signs of foundation damage.

**ST 151 K** - Installed in 1963, 137 ft water depth. Survived Andrew. The similar ST 151 H platform which toppled in Andrew and was connected by bridge to ST 151 K. Waves were known to impact the deck of this platform during Andrew. The inspection revealed several flooded members, however, further investigation of these members did

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not reveal any damage that appeared to be caused by Andrew or would affect the platform's structural integrity. In fact, the platform appeared in remarkably good condition considering the platform's age and the level of loading it sustained in Andrew.

**ST 130 Q** - Installed in 1964, 170 ft water depth. Survived Andrew. Waves were known to impact the deck of this platform during Andrew. This is also a quarters platform resulting in a large projected area for wind loads. The inspection revealed several flooded members located near the waterline, including a vertical diagonal brace that was heavily loaded during Andrew. Further investigation of the brace revealed a crack extending perpendicularly from one of the joints. The crack did not appear to be completely through the member wall since the member was not flooded in this region. The experienced diver performing the inspection indicated that this appeared to be a "laminar" crack often seen offshore that is the result of faulty fabrication when the platform was originally constructed. It does not appear that this defect significantly decreases the platform's capacity, although the crack should be monitored to determine if there is any growth. Similar to ST 151 K, this platform was otherwise in good condition considering its age and loadings experienced in Andrew.

### Analysis

Following the inspections, each of the platforms was analyzed using the nonlinear computer code CAP (Capacity Analysis Program), developed by PMB and also owned by the MMS and several other operators. Computer models originally developed for the Andrew JIP were updated to reflect actual field conditions of the platforms (e.g. proper number of boat landings) as well as improvements to the analytical modeling of joints and the foundation.

The improved joint modeling included revised joint capacity based upon the best information available in the public domain and a modified force deformation characteristics to more properly reflect the types of failure modes expected of the joints. Of particular importance was the incorporation of elastic-brittle joint modelling based upon study of the similar ST 177 B platform which was severely damaged in Andrew with multiple joint failures, and also studied for the MMS (PMB, 1994d).

The improved foundation modeling consisted of a 100 percent increase in soils strength over that determined per API RP 2A to account for conservatism in the API equations (also based upon study of the ST 177 B platform, PMB, 1994).

Analysis results for the toppled ST 130 A platform provided the most information and it was found that the analysis could be "tuned" so that the analysis results match observed results. In this case, it was determined that the computer model with brittle joints, a foundation modeled

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using increased (100%) soil strength, and a localized, ungrouted pile-leg annulus (reflecting a poor grout job during the original platform installation) predicts a platform failure remarkably similar to that observed during Andrew.

Results for the ST 130A and ST 130 Q platform were less informative, partly because the platforms exhibited no damage that could be directly compared to that predicted by the computer model. The joint and foundation modeling improvements did increase the level of loading required to fail the platform; however, the computer models still predicted substantial damage to the platform's joints and foundation, yet no damage was observed during the inspections.

Thus, similar to the Andrew JIP, the computer models of this study would also label these platforms as unexpected survivals. One possible explanation is wave loads on the deck. The analytical model assumes full impact of the wave across the entire deck resulting in large deck loads. However, discussion with Chevron indicates that based upon observed damage to the deck, waves did impact the deck, but only in certain areas, and most likely not across the entire width. If this was accounted for in the computer model, the total loads acting on the platform would reduce significantly and perhaps the computer model would predict little or no damage to the jacket as was observed in the inspections.

## Conclusions and Recommendations

Overall, the detailed inspections conducted by this study confirmed that the visual inspections performed by operators immediately following Andrew and in response to MMS NTL 92-07 provided a good assessment of the general condition of the platforms. The detailed visual and MPI inspections performed for this did not reveal any other significant damage resulting from Andrew. However, this study investigated only three platforms of similar vintage and design all owned by the same operator. The results may differ for different operators and different types of platforms.

At this time it appears that computer analysis to that used in this project can be used to identify platforms that may be at risk in large hurricanes. This study confirms that the ultimate capacity approach recommended in API RP 2A Draft Section 17 provides a reasonable estimate of platform performance and is appropriate for use in assessment of existing platforms.

This study, as well as the Andrew JIP, have shown that if performed properly, these types of analysis are generally conservative. However, further work is recommended, and in some cases, is ongoing to refine these types of analysis so that the predictions of platform performance can be more accurate. Recommendations include:

- Joints. The most significant variation appears to be joint capacity and joint modeling.

Further studies are needed that address both joint capacity and modeling such as the proposed Andrew JIP Phase II (PMB, 1994b) and the three dimensional Frames Test in the UK proposed by Bomel.

- Foundations. There appears to be conservatism in the API RP 2A formulations which are typically used by operators to estimate foundation capacity. The ongoing API/MMS (PMB, 1994a) study of foundation behavior during Andrew should help address this issue particularly since some of the study focuses upon observed foundation failures of caissons. Further observations and associated analytical evaluation of a platform that experienced a foundation failure during Andrew or another large storm would be most helpful.

- Wave-in-Deck. For the wave-in-deck condition, it appears that current approaches to estimate the deck loads may be too conservative, since the wave does not always impact the full length of the deck in a "long crested" fashion as assumed in the analytical models. However, for platform assessment-type work, it is conservative to make this assumption and is the best approach until further information and field observations or realistic model testing of waves impacting decks is available.

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## **SECTION 1 INTRODUCTION**

### **1.1 Background**

Hurricane Andrew passed through a heavily populated region of offshore platforms during August 1992. Andrew was a severe hurricane with wave heights reaching 60-70 feet in some regions. While most platforms survived Andrew with little or no damage, several platforms were severely damaged, and in some cases failed.

The Minerals Management Service (MMS) subsequently contracted PMB to perform follow-up work related to platforms affected by Andrew. There were two main tasks:

**1. Platform Inspections.** Perform underwater inspections of several platforms, focusing on possible damage caused by Andrew, such as broken braces or damaged joints. Where possible, try to ascertain the failure mechanism of the member (e.g. buckling, bending, joint punching) or platform (foundation failure, multiple brace failure).

**2. Platform Analysis.** Using the results of the underwater inspection, perform nonlinear pushover analysis to determine if the observed damage (or lack of damage) can be predicted analytically. Make appropriate recommendations that can be used in the future to improve the capability of this type of analysis to accurately predict platform performance.

This reports presents the approach, results and conclusions associated with each of these tasks. Additional detailed information regarding the platform inspections can be found in a separate document (Appendix A).

### **1.2 Acknowledgements**

The MMS and PMB appreciate the assistance Chevron in providing the opportunity to inspect several Chevron platforms for use in the project. Particular thanks go to Mr. Dirceu Botelho of Chevron who was instrumental in providing access to the platforms in addition to drawings and other pertinent information necessary for both the inspection and analysis work.

### **1.3 Contract Information**

This work was performed for the MMS, as Task 1 of the Architect-Engineering Services Contract No. 1435-0001-30700. The work was performed during the period from September 1993 to June 1994.

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## **SECTION 2                      PLATFORM INSPECTIONS**

This section summarizes the results of the underwater inspections, including the more important and interesting inspection drawings and photos. A completed detailed summary of the inspection including additional photos and a video is included in a separate document (Appendix A).

### **2.1 Platform Selection**

Platforms were selected based upon results of the Andrew JIP which provided key information such as drawings, structural analysis and prior post-storm inspection information on 13 platforms affected by Andrew (survived, damaged, failed). The platforms contributed to the JIP by Chevron were of particular interest since they were located in the South Timbalier area which was a region of large waves. Chevron also had a mix of platforms that survived, were damaged, or were toppled in Andrew.

Negotiations were then undertaken with Chevron to select possible platforms. It was decided that three platforms would be inspected:

1. Platform ST 130 A which toppled during Andrew and had not yet been salvaged. Figure 2-1 shows the general configuration of this eight leg, 140' water depth platform. The toppled structure was still relatively intact, laying on its side, providing a good opportunity to investigate why the platform failed.
2. Platform ST 151 K which survived Andrew, apparently undamaged per Chevron's post-storm inspections. Figure 2-2 and Photo 2-1 show the general configuration of this eight leg, 137' water depth platform. ST 151 K was determined during the Andrew JIP to be an "unexpected" survival since the JIP risk analysis indicated that the platform should have suffered significant damage. In addition, a sister platform ST 151 H, which was connected to ST 151 K by bridge, toppled during the storm. Thus based upon computer analysis and observed performance of ST 151 H, this platform should have been damaged in Andrew - but no damage had yet been found. There was therefore an opportunity to possibly uncover some additional damage to the platform that may have been missed during previous visual only inspections. The JIP computer analysis results were used to help focus the inspections on heavily loaded platform members.
3. Platform ST 130 Q which also survived Andrew, apparently undamaged. Figure 2-3 and Photo 2-2 show the general configuration of this four leg, 170' water depth platform. This is a similar situation to the ST 151 K platform - an unexpected survival. In addition, it provided an opportunity to inspect a different type of structure in that this was a four leg (vs. eight leg) structure and also a quarters platform (vs. well head



platform).

### 2.2 Inspection Plan

An inspection plan was developed for each of the platforms using analysis results from the Andrew JIP combined with supplemental information made available from Chevron regarding their previous visual inspections of the platforms. Key issues were as follows:

**1. Flooded Member Checks.** All of the platform members were fabricate to be buoyant, thus a flooded member indicates that the integrity of the member has been compromised with a crack, tear or other type of through-hole that allows water to enter. In some cases, the flooding may be the result of rainwater that accumulates in the member during fabrication. All primary members in the surviving platforms were tested for flooding in order to provide a preliminary indication of damage such as cracks or tears. Secondary bracing related to conductor framing, boat landings, anodes, etc. was neglected. Photo 2-3 shows the ultrasonic device used to check for flooded members. The device is easy to use and is simply placed by the diver on the member of interest while an operator on the surface takes a reading to determine if the member is flooded. In some cases the diver may have to scrape away marine growth to ensure good contact with the member.

**2. Joint Inspections.** A previous MMS sponsored on-shore inspection of the salvaged ST 177 B platform (PMB, 1994), which has a similar bracing scheme as the platforms in this study, indicated problems with the X, K and KT joints. The X joints indicated large cracks along the tension member, the K joints indicated complete sever of the chord, and the KT joints indicate bulges in the chord. These particular failure modes were checked on the surviving ST 151 K and ST 130 Q platforms by selecting the most heavily loaded and most accessible joints for cleaning, followed by a close visual and Magnetic Particle Inspection (MPI) for crack detection. The chord of the K joints were also tested for bending and bulges using a 3 ft long flat piece of metal. Photo 2-4 shows the MPI magnet. Photo 2-5 shows an MPI in progress. A special colored dye containing steel filings is placed on the area of interest (e.g. weld). The MPI magnet is then placed across the area (e.g. weld) and energized by an operator on the surface. The result is easy identification of any cracks or other surface defects as the magnetized particles are aligned along the defective region.

**3. Most Heavily Loaded Member(s).** The most heavily loaded one to two members of the platform during Hurricane Andrew were checked for cracks at the leg-brace weld. The members were identified based upon the previous analysis performed in the Andrew

JIP. MPI was used to check for cracks not visible to the naked eye.

**4. Popped Marine Growth.** Chevron reported that on one of the platforms, an underwater inspection immediately following Andrew indicated that an area of marine growth appeared to "pop" away from the member, indicating that perhaps some plastic deformation had occurred. A similar circumstance was reported by Amoco in the inspection of one of their post-Andrew platform inspections (Imm, G., et.al., 1994). Amoco later determined that some plastic deformation had indeed occurred at the joint. For this project, the suspect member was cleaned, visually inspected and then MPI was used to check for cracks.

### **2.3 ST 130 A Inspection**

This was the most difficult platform to inspect since it collapsed in Andrew with no visible debris located above the waterline. This made it difficult to first locate the platform as well as stay on location during the inspection. Figure 2-6 shows a rendering of the collapsed platform based upon previous inspections by Chevron.

The dive vessel arrived at the platform site using the satellite based Global Positioning System. A weighted block attached to a buoy was placed at the assumed platform location. The dive vessel then moved off site approximately 200 feet and a mesotech sonar device mounted on a tripod was then lowered to the seafloor to scan for debris. Debris were not found on the first pass. A diver was then sent down the buoy line and began a radial search for debris. The deck crane was first located, with the diver following this to the deck and remainder of the platform. A 2 1/2 inch nylon line was then tied to a leg of the platform and used to moor the dive vessel.

The mesotech device was then repositioned to gain a clearer image of the platform debris as shown in Photo 2-5. This matched well with the rendering from the previous inspection shown in Figure 2-4.

A total of 5 dives were made on this platform using visual inspections and photographs. A video was also taken of the toppled platform (this was the only platform that was video taped). Due to high currents and the unusual mooring configuration of the dive vessel, it was decided to not "penetrate" the debris for closer inspection. Therefore, inspection was limited to the platform framing closest to the water surface (i.e. along Row 1) plus inspection of the remaining base section. The primary results of the inspection were as follows:

- The platform fell to the northwest, in basically a single piece, with the break occurring at the second bay up from the mudline. See Figure 2-3. Note that Chevron had replaced the deck legs and deck of the platform in approximately 1990 and that these items,

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designed to newer design codes, appeared to be in very good condition.

- There was damage to the upper level K and x braces, although not of the magnitude seen in the on-shore inspection of the ST 177 B platform also performed for the MMS (PMB, 1994d). The damage consisted of indentations and bulges at the brace-chord intersections. There were no shear failures as seen at ST 177 B, although only Platform Row 1, closest to the water surface, was investigated.

- A detailed inspection of the mudline at Leg D2 showed no evidence of foundation (pile) failure. The base appeared reasonably level with the horizontal members and mudmats in contact with the seafloor providing no clear evidence of pile pull-out or plunging. There was also no evidence of "gaping" between the leg and soil, which would have indicated large horizontal pile displacement, although the piles themselves were not visible. Note that the inspection was performed over a year after Andrew and it is possible that any "gaps" in the soil would have been filled by action from waves and currents.

- The platform appeared to topple due to bending failure of the legs at approximately elevation -95', as shown in Photos 2-7 through 2-10. These photos indicate a clear bending failure as evidenced by the ovalized leg (33" dia. by 0.50" thick) and pile (30" dia. by 0.75" thick). The positioning of the leg ovalization indicates that the platform toppled directly to the north (i.e. the leg was ovalized in a vertical direction), yet the main section of the debris was located to the northwest. One explanation is that the platform began to topple to the north and that current and wave forces pushed the debris in the northwest direction as it fell.

The pile/leg annulus was supposedly grouted which provides for a composite leg-pile resulting in a stronger structural cross-section, particularly in bending. However, photos (Photos 2-7 through 2-10) and video indicated no clear evidence of any grout in this region of the leg. The divers were specifically asked to look for grout or grout fragments yet none was found. Discussions with Chevron indicate that previous independent inspections by Chevron had determined that perhaps there was grout in this area. As noted above for soil gaps, it is possible that previous storms had washed away any traces of the grout. As described later, an analysis of the platform assuming missing grout in this region of the leg closely matches the observed failure mode of the platform.

- There were no indications of pre-existing platform damage such as dents, cracks, holes or excessive corrosion that would have lead to the platform failure. The platform appeared to be maintained in a manner typical of Gulf of Mexico operations.

## **2.4 ST 151 K Inspection**

The ST 151 K platform inspection was similar to performed on a typical offshore platform, with the dive vessel moored to the platform and dive operations undertaken over a period of 1-2 days.

The inspection extended beyond a typical API Level II category since a complete flooded member check was performed and several joints were cleaned and examined with MPI. As previously noted, prior computer analysis conducted during the Andrew JIP was used to determine which joints to clean and inspect with MPI. Additional joints were also cleaned and inspected with MPI when a member was found to be flooded. This is also the structure that had reports of "popped" marine growth and this region of the platform was cleaned and inspected with MPI.

The results of the inspection are shown in Figure 2-5 which shows all of the vertical and horizontal platform members and the results for inspections on each. Only primary member framing is shown. A member was found to be nonflooded unless indicated otherwise. Primary results of the inspection are as follows:

- All primary members were first checked for flooding. A total of approximately 120 members were checked with 9 members found flooded. Several of these primary flooded members were further inspected as noted below.
- Joint inspections Row C. The X, uppermost K and KT joints on Row C were cleaned and MPI inspected based upon results of the Andrew JIP computer analysis and upon the on-shore inspection of the similar ST 177 B platform. The computer analyses indicated that these inner transverse row joints (closest to the conductors) were some of the most highly loaded during Andrew, and the ST 177 B inspection indicated that these joints were found to be severally damaged. As indicated in Photos 2-11 through 2-13, close visual and MPI showed no damage to these joints. Photo 2-14 shows the straight bar that was placed on top of the joint to determine if it had deflected upward due to high loads in the braces. There were also no indications of damage of any kind using this method.
- Joint inspection Leg C2 Elev. -63 ft. This was the area that was reported to have "popped" marine growth based upon a previous visual inspection by Chevron. The area was cleaned and MPI performed with no damage found (Photo 2-15).
- Inspection of member VD A2 -134' to B2 -100'. This is a major vertical diagonal located near the mudline that was heavily loaded in compression during Andrew. The member was found to be partially flooded and therefore further inspected since the damage may have resulted from Andrew. The member was not flooded near the -100'

elevation, indicating that the joint at this level was still fully intact. Further inspection was therefore focused on the lower -134 joint and along the length member. The -134' was cleaned and MPI with no damage found (Photo 2-16). The member was also checked with close visual inspections long its length with no apparent damage found. The connection point along the member of the original anode was also cleaned and MPI with no damage found. It is unclear why the member was flooded.

- Inspection of member VD C1 -134 ft. to D2 -100 ft. This is also a major vertical diagonal located near the mudline that was heavily loaded in tension during Andrew. A similar inspection approach was used as described above for the other flooded vertical diagonal. The inspection again showed no signs of damage to the member with no clear reason why the member was flooded.

- Inspection of horizontal member A1 to B1 -63'. This is a horizontal that was loaded in compression during Andrew and was found to be flooded. Both ends of the member were cleaned. The inspection revealed that a segment of the weld at B1 was not completed during original fabrication, as shown in Figure 2-6 and Photo 2-17, resulting in a path for water to enter and flood the member. The flooding was therefore not caused by Andrew. This anomaly has been in place since the platform was originally installed and is not expected to significantly impact the structural integrity of the platform.

- Several of the flooded members were horizontals located near the mudline. Flooded mudline horizontal are common due to localized damage at the joints due to excessive vibrations caused by pile driving. Thus damage to these members is most likely not associated with Andrew. Fortunately these mudline horizontal members do not contribute significantly to platform strength.

- Flooded horizontals -63' and -26'. These are flooded horizontals that cross through the region of the conductors with numerous secondary conductor support framing tied back to the member. These flooded members were not checked further since the flooding is likely caused by small cracks at the locations of secondary conductor guide support framing. These cracks do not significantly impact the global strength of the platform.

- Flooded VD A1 -26' to B1. This member was not inspected further due to lack of time.

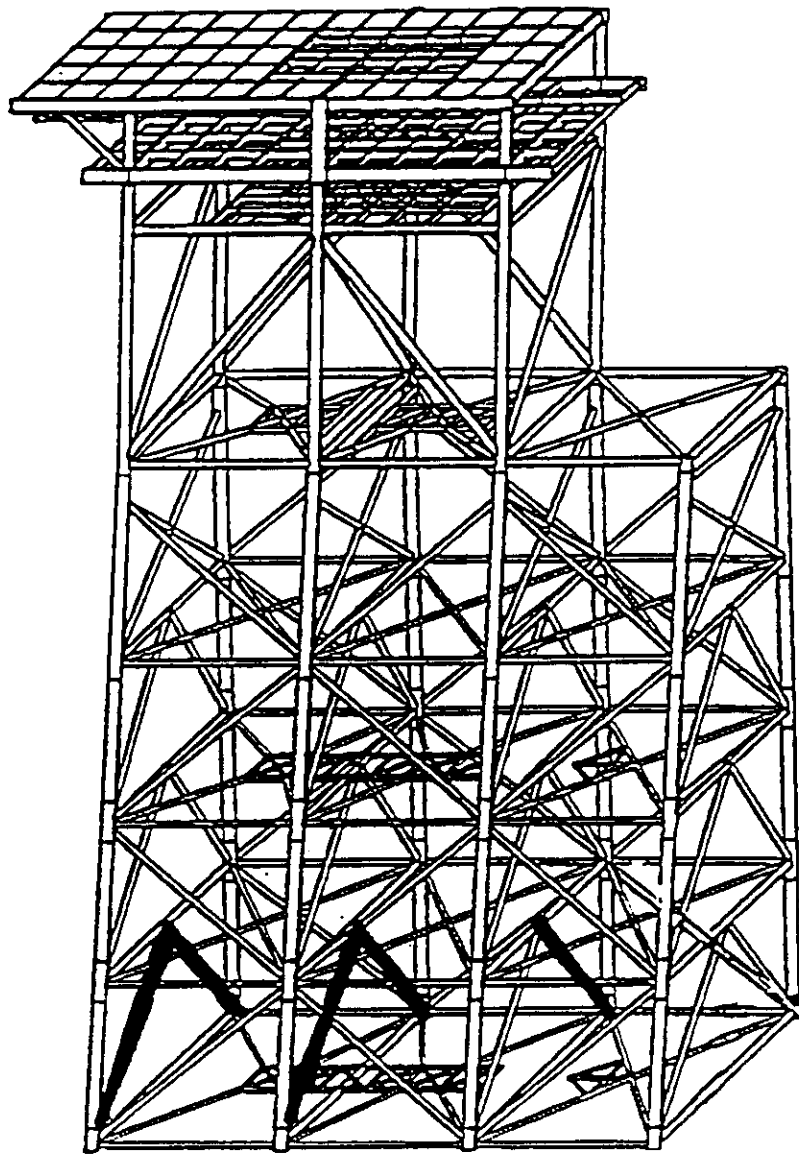
## **2.5 ST 130 Q Inspection**

The ST 130 Q platform inspection was similar to that performed on the ST 151 K platform. The inspection consisted of visual inspections combined with flooded member checks and cleaning/MPI of selected joints.

The results of the inspection are shown in Figure 2-7 which shows all of the vertical and horizontal platform members and the results for inspections on each. Only primary member framing is shown. A member was found to be nonflooded unless indicated otherwise. Primary results of the inspection are as follows:

- All primary members were first checked for flooding. A total of approximately 70 members were checked with 5 members found flooded. Several of the flooded members were horizontal located near the mudline. Flooded mudline horizontal are common due to damage at the joints during pile driving. Thus damage to these members is most likely not associated with Andrew. Fortunately these mudline horizontal members do not contribute significantly to platform strength. Several of these flooded members were further inspected as noted below. The interior horizontal X braces were not checked due to lack of time (these members are not expected to play a key role in the platform's structural integrity).
- Joint inspections Row 2. The X, and uppermost K joint on Row C were cleaned and MPI inspected based upon results of the Andrew JIP computer analysis. The computer analysis indicated that these joints were some of the most highly loaded during Andrew. As indicated in Photos 2-18 and 2-19, close visual and MPI inspection showed no damage to these joints. The flat metal bar check also showed no signs of damage to the joints.
- Inspection of member VD A1 -170' to midpoint A1/A2 -130'. This is a major vertical diagonal located near the mudline that was heavily loaded in compression during Andrew. The member was found to be partially flooded (not flooded at -130') and therefore further inspected since the damage may have resulted from Andrew. Both ends of the member were cleaned and inspected with MPI with no damage found. An anode support connected to the member was also cleaned and MPI with no indication found. It is unclear why the member was flooded.
- Inspection of member VD A1 -170' to midpoint A1/A2 -130'. This is a major vertical diagonal located near the mudline that was heavily loaded in tension during Andrew. The member was partially flooded (not flooded at -130') and therefore was further inspected since the damage may have resulted from Andrew. Due to time limitations,

only the upper joint of the member was cleaned. The close visual inspection and NDT found a 4 in. indication radiating down the member (Figures 2-8 and 2-20). The crack progressed into the uncleaned portion of the member where it eventually disappeared into the marine growth. The inspection diver noted that the crack looked similar to other "laminar" cracks noticed on other projects. A laminar crack is a crack that appears in one of the layers of steel used to make the tubular. Laminar cracks are often not a through crack since they extend through only a single layer of the steel. As noted above, the member was not flooded in this region, indicating that this is not a through crack and that a "laminar" crack may indeed be the case. Overall, there was no clear reason why the member was flooded. Since the crack is perpendicular to the joint of this primarily axially loaded member, it is not anticipated that this crack will significantly impact the platform strength. However, it is advisable to monitor this crack in the future to look for further growth or an indication that the member is flooded in this region, indicating that the crack may have progressed to the through condition.

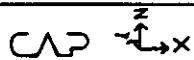
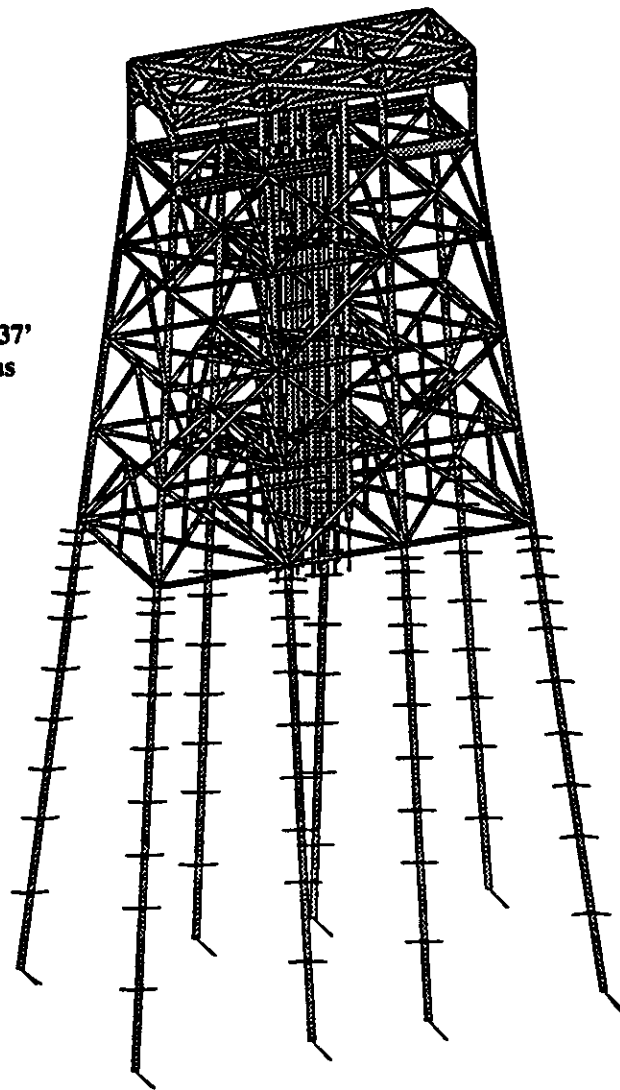


**Figure 2-1 Platform ST 130 A - Toppled in Andrew**



**Platform ST151K**

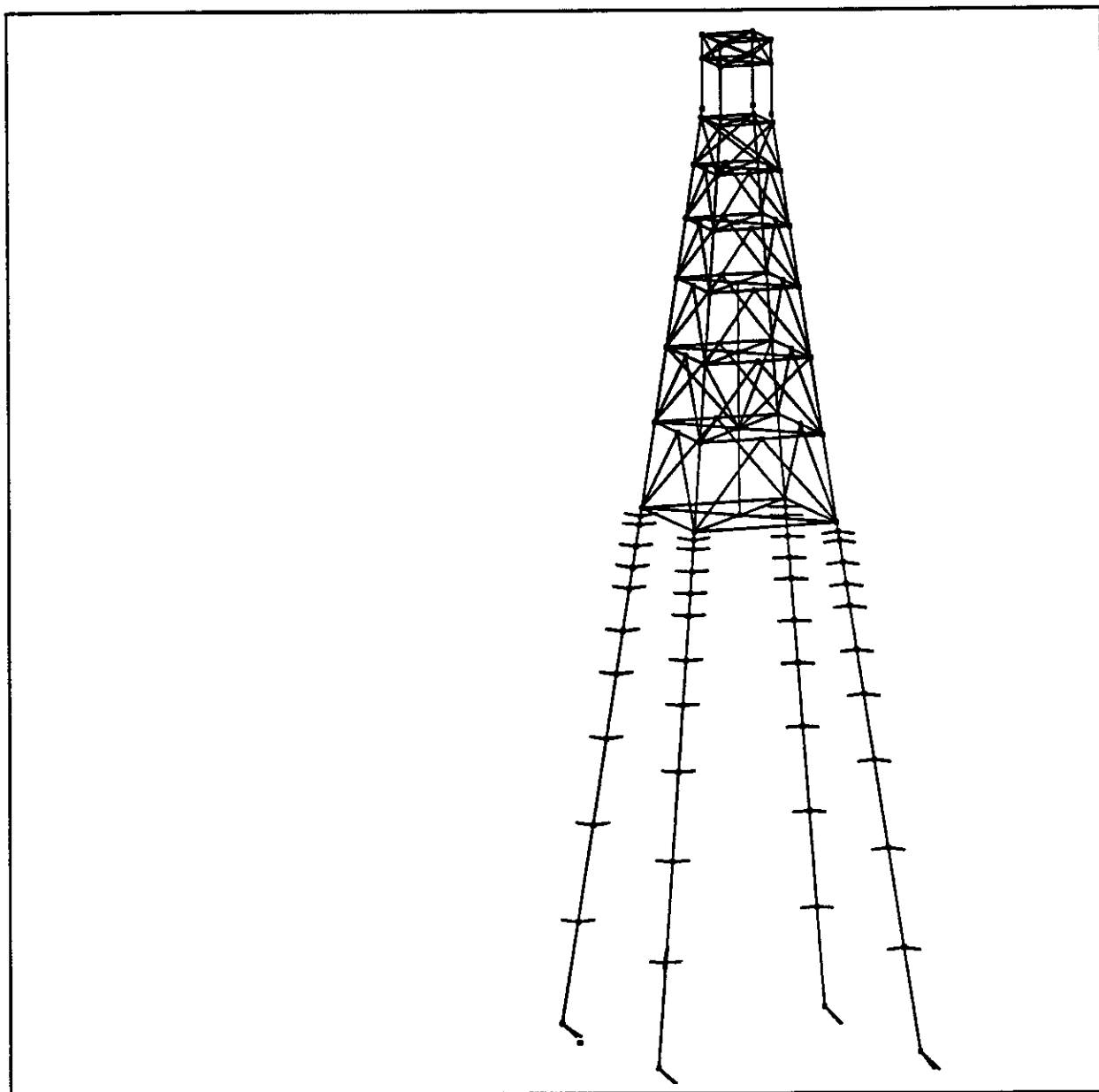
- Survived Andrew
- Year Installed - 1963
- Water depth = 137'
- 12 - 30" Dia. Conductors
- Cellar Deck Elevation = 37'
- Grouted Leg-Pile Annulus



Chevron ST151K - Analytical Model

**Figure 2-2 Platform ST 151 K - Survived Andrew**

Project: ST130Q Model



CAP  Model View

**Figure 2-3 Platform ST 130 Q - Survived Andrew**

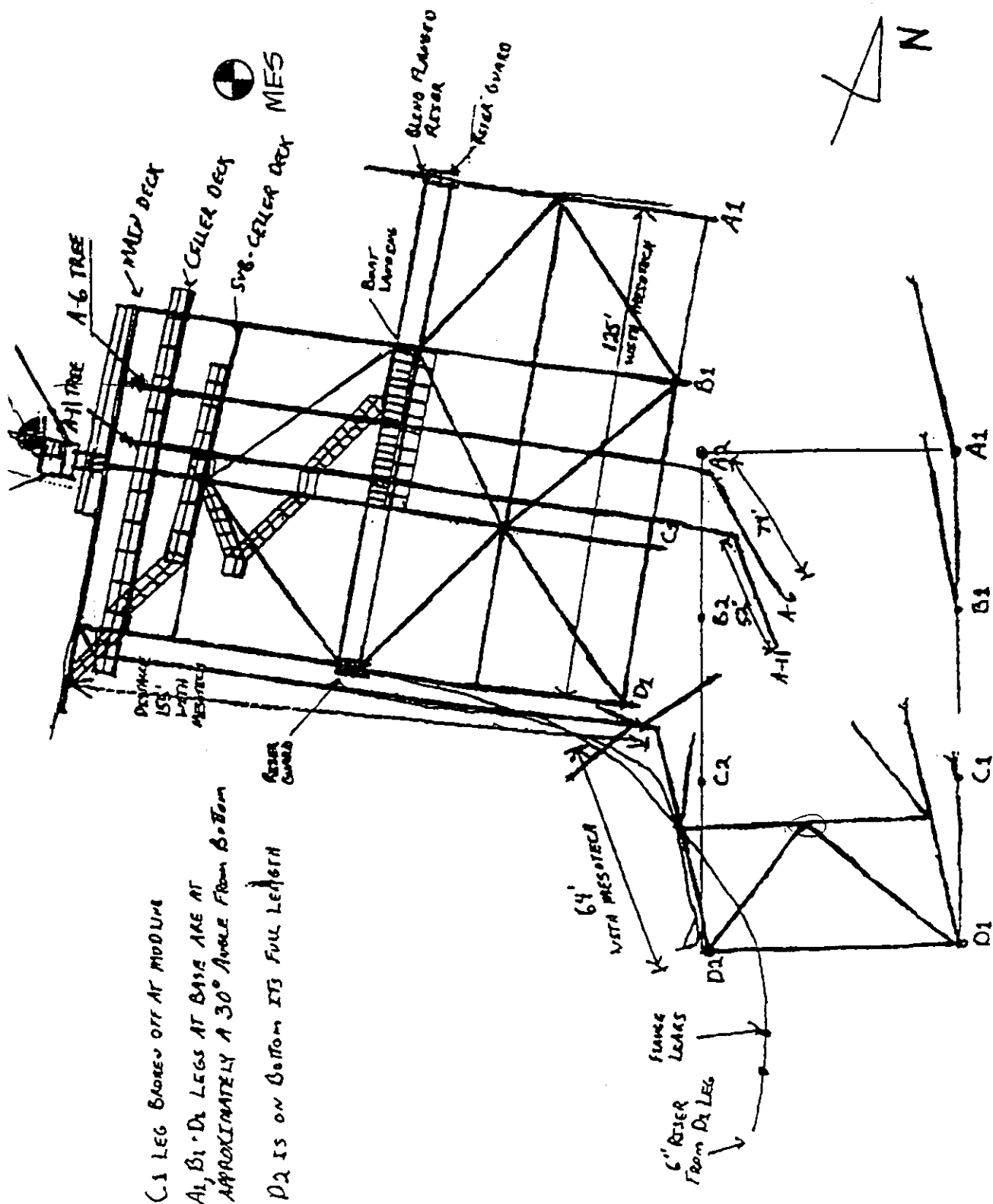
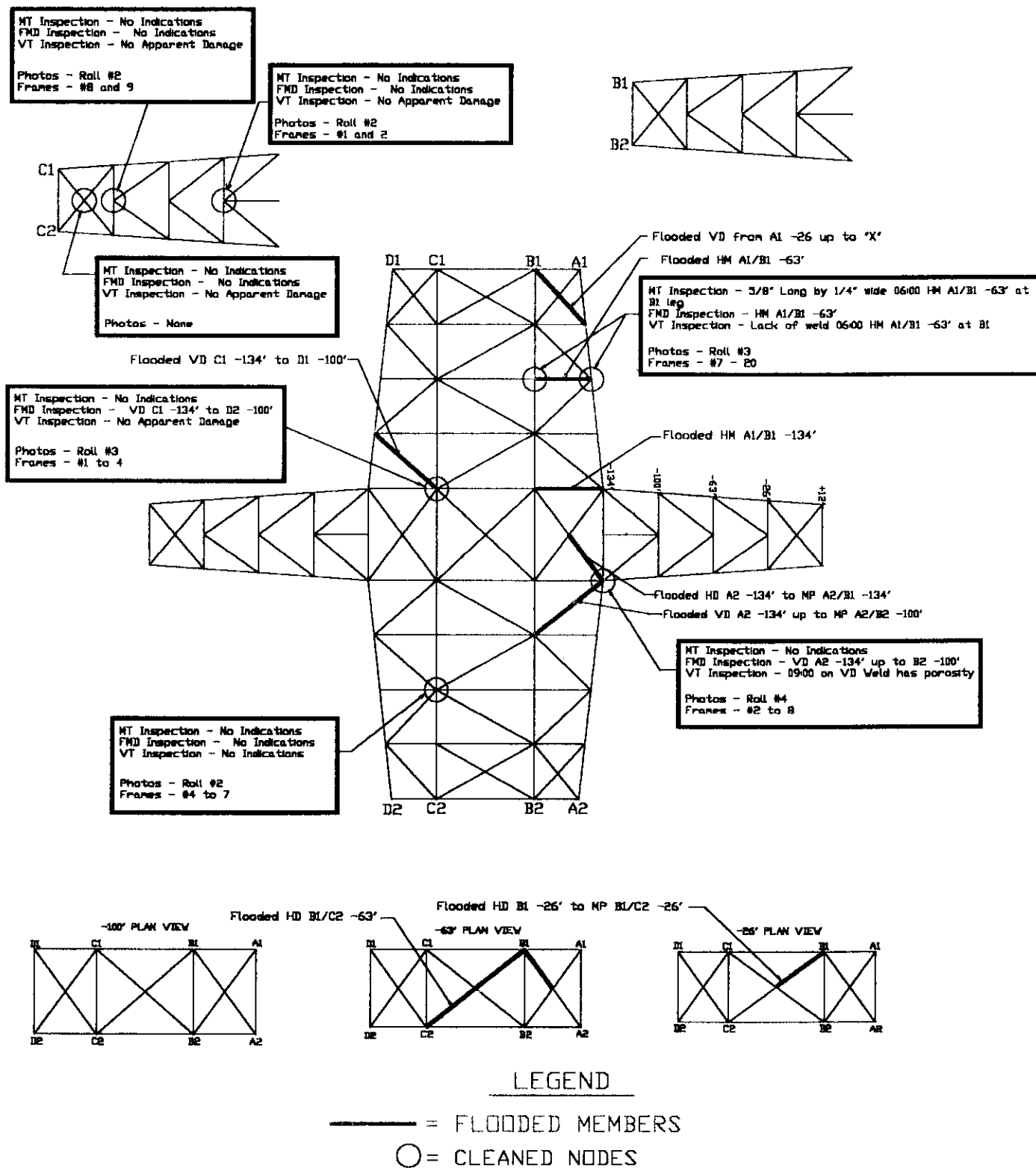


Figure 2-4 Diver Drawing of the Toppled ST 130 A Platform

**Figure 2-5 Platform ST 151 K - Inspection Results**



INSPECTION NO. 93/072	CLIENT PMB	INSPECTION RESULTS SUMMARY	DWG. NO. ST151K (FIG 1)
DATE 11/16/93	STRUCTURE ST-151-K	ST-151-K	OCEANEERING

# MAGNETIC PARTICLE INSPECTION REPORT

Client's Ref. Number : 88967-312

File H:\WPWIN6\WPDOS\MTST151K.WK4

©Edwards

OI Job Number : 15855

OI Inspection Number : 93/072

Date : 11/17/93

Form Number MT- ST151K #1

Reference Form Number :

Page : 1 of 1

## GENERAL INFORMATION

Client : PMB/BECHTEL

Contractor : OCEANEERING INT.

Cert.

Client's Representative : F. PUSKAR

Underwater Inspector : B. TORLINE

MT-II

Structure ID : ST-151 K

Topside Inspector : K. EDWARDS

MT-III

Joint/Weld ID : HM A1/B1 -64' @B1

Diving Supervisor : K. EDWARDS

## VISUAL INSPECTION RESULTS

Visual Indication : Y (Yes)(No)

Indication Location Weld : B Tow Haz Bead B Brace Chord

Clock Position : 6 To 6

Est. Width : > (>X<) Hairline <OR>

Visual Length : 5/8"

Visual Width : 1/4"

Visual Material Separation : Y (Yes)(No)

Visual Depth : .5 "

## MAGNETIC PARTICLE INSPECTION RESULTS

MT Indication : Y (Yes)(No)

MT Indication ID Number : HM A1/B1 -64 @B1

Clock Position : 6 To 6

Indication's Location Weld : B Tow Haz Bead B Brace Chord

MT Length : 5/8 Inches (Meas.) (Est.)

Orientation : L Longitudinal Transverse Both

Particle Build-up : H Heavy Moderate Light

Confirmatory Grinding : N (Yes)(No)

Particle Adhesion : M Strong Moderate Weak

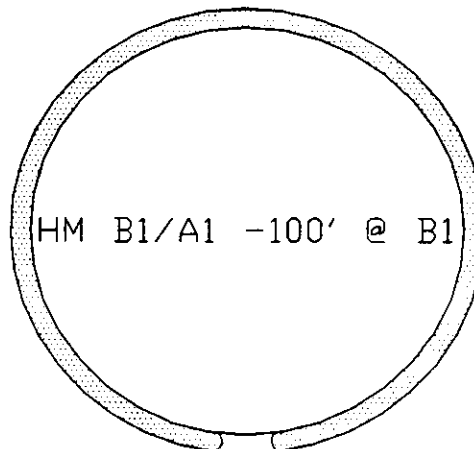
Re-Test Indication : N (Yes)(No)

Depth Estimate on Particle Appearance : >1/16 <or> 1/16"

Remedial Grinding : N (Yes)(No)

## DRAWINGS AND NOTES

LEG B1



Area where there is Lack of weld metal  
5/8" Long by 1/4" Wide

Signature U/W Inspector :

*B. Torline*

Signature T/S Inspector :

*Robert Edwards*

## FINAL EVALUATION

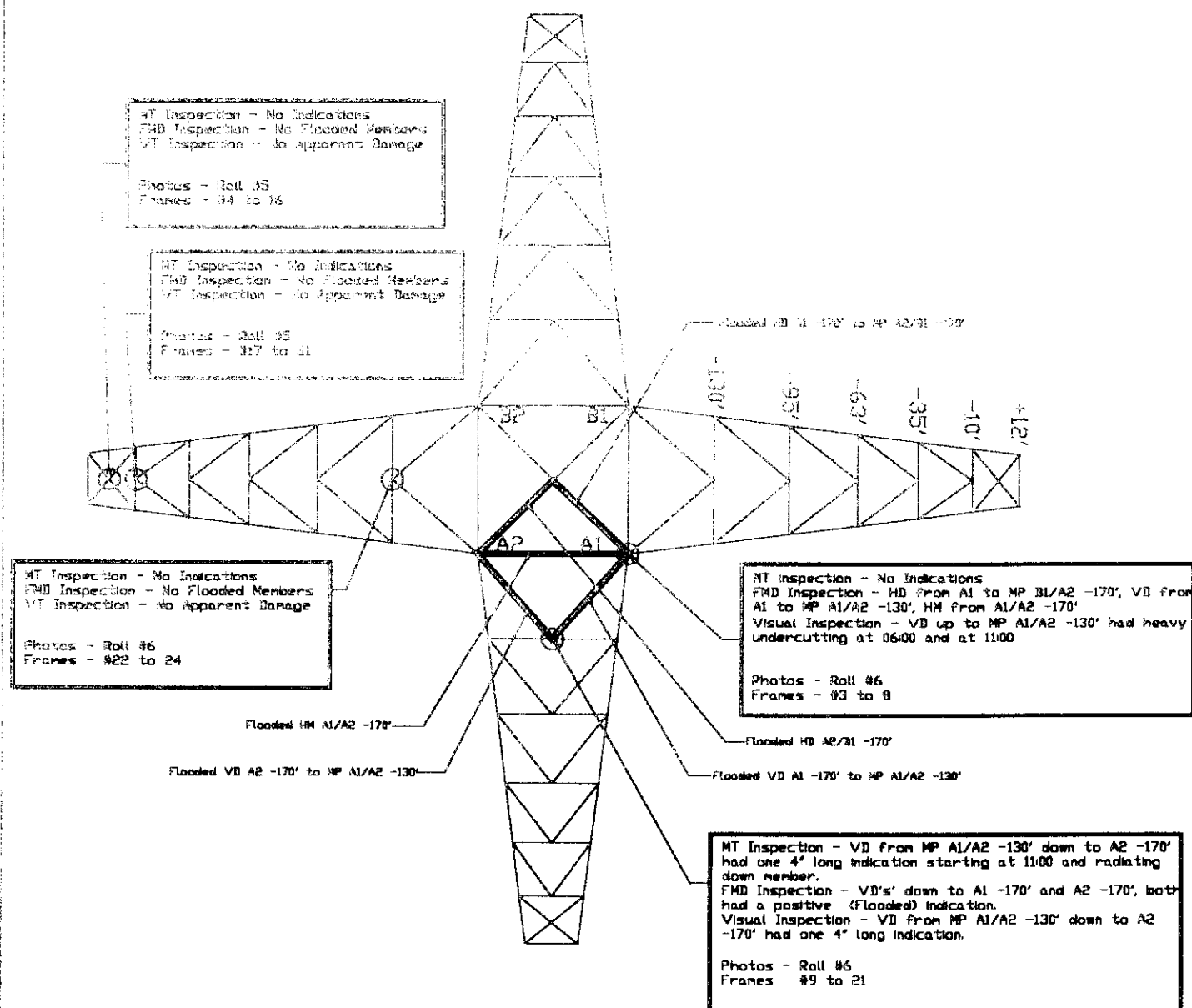
Final Evaluation : THE 06:00 POSITION OF THIS HM HAD A TEMPORARY CONSTRUCTION PADDED INSTALLED AT THIS

LOCATION. THERE IS A LACK OF WELD MATERIAL . THE HM A1/B1 -64' WAS FLOODED

## DOCUMENTATION

Figure 2-6 Platform ST 151 K - Missing Weld Material

**Figure 2-7 Platform ST 130 Q - Inspection Results**



Legend

— = Flooded Member  
○ = Cleaned Nodes

DWG SCALE N/A	CLIENT PMB	INSPECTION RESULTS SUMMARY ST-130-QUARTERS	DWG. NO. ST130Q (FIG 2)
DATE 11/18/93	STRUCTURE ST-130-Q		

# MAGNETIC PARTICLE INSPECTION REPORT

Client's Ref. Number : 88967-312

File H:\WPWIN60\WPDOS\MTST130Q.WK4

©Edwards

OI Job Number : 15855

OI Inspection Number : 93/071

Date : 11/18/93

Form Number MT- ST130Q #1

Reference Form Number :

Page : 1 of 1

## GENERAL INFORMATION

Client : PMB/BECHTEL

Contractor : OCEANEERING INT.

Cert.

Client's Representative : F. PUSKAR

Underwater Inspector : B. TORLINE

MT-II

Structure ID : ST-130Q CHEVRON

Topside Inspector : K. EDWARDS

MT-III

Joint/Weld ID : VD MP A1/A2 -100' TO A2 -130'

Diving Supervisor : K. EDWARDS

## VISUAL INSPECTION RESULTS

Visual Indication : Y (Yes)(No)

Indication Location Weld : B Tow Haz Bead B Brace Chord

Clock Position : 11 To 11

Est. Width : > (>X<) Hairline <OR>

Visual Length : 4"

Visual Width : 1/64"

Visual Material Separation : N (Yes)(No)

Visual Depth : NA

## MAGNETIC PARTICLE INSPECTION RESULTS

MT Indication : Y (Yes)(No)

MT Indication ID Number : VD MP A1/A2 -100' TO A2 -130'

Clock Position : 11 To 11

Indication's Location Weld : B Tow Haz Bead B Brace Chord

MT Length : 4 Inches (Meas.) (Est.)

Orientation : T Longitudinal Transverse Both

Particle Build-up : H Heavy Moderate Light

Confirmatory Grinding : N (Yes)(No)

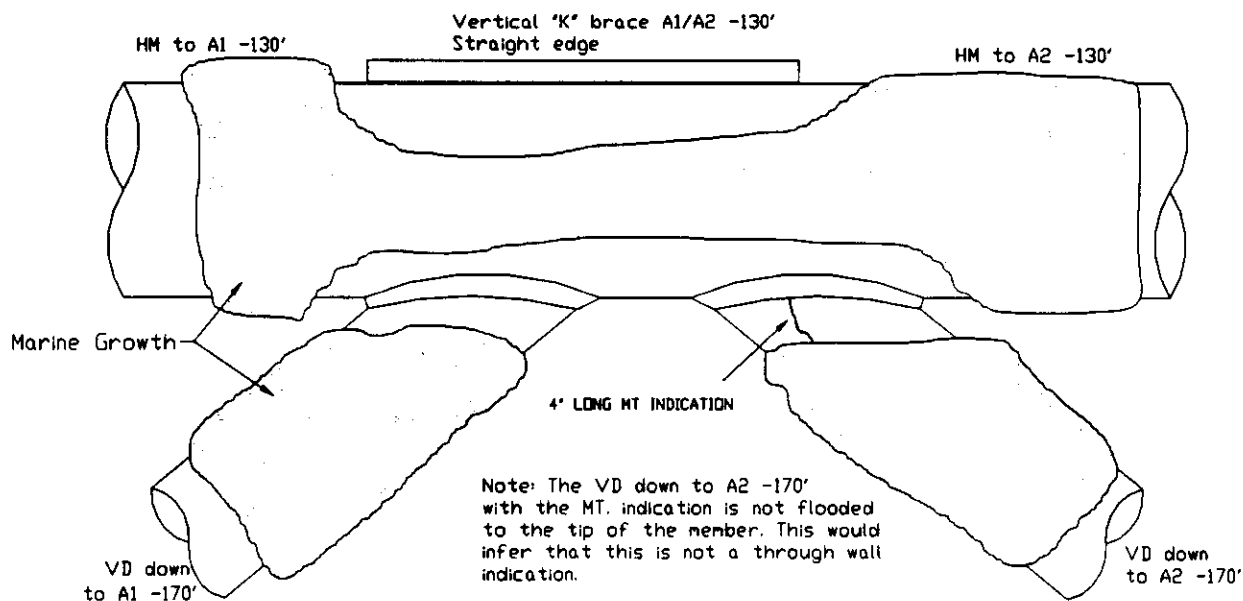
Particle Adhesion : S Strong Moderate Weak

Re-Test Indication : N (Yes)(No)

Depth Estimate on Particle Appearance : <1/16 < or > 1/16"

Remedial Grinding : N (Yes)(No)

## DRAWINGS AND NOTES



Signature U/W Inspector :

*B. Torline*

Signature T/S Inspector :

*Kenneth Edwards*

## FINAL EVALUATION

Final Evaluation : A TRANSVERSE INDICATION STARTING AT 11:00 IN THE HAZ RADIATING DOWN THE MEMBER

THIS MEMBER WAS NOT COMPLETELY FLOODED INFERRING THAT THE INDICATION WAS NOT THROUGH WALL

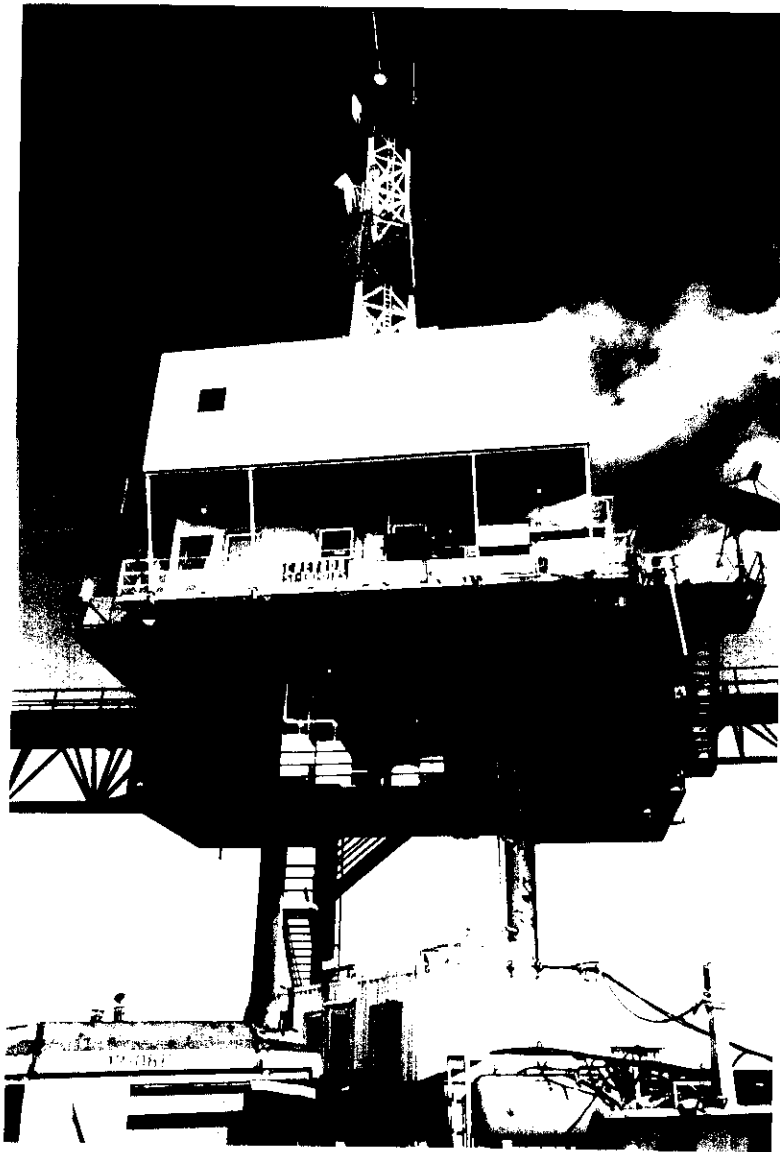
## DOCUMENTATION

Figure 2-8 Platform ST 130 Q - Crack in Member



**Photo 2-1 Platform ST 151 K**

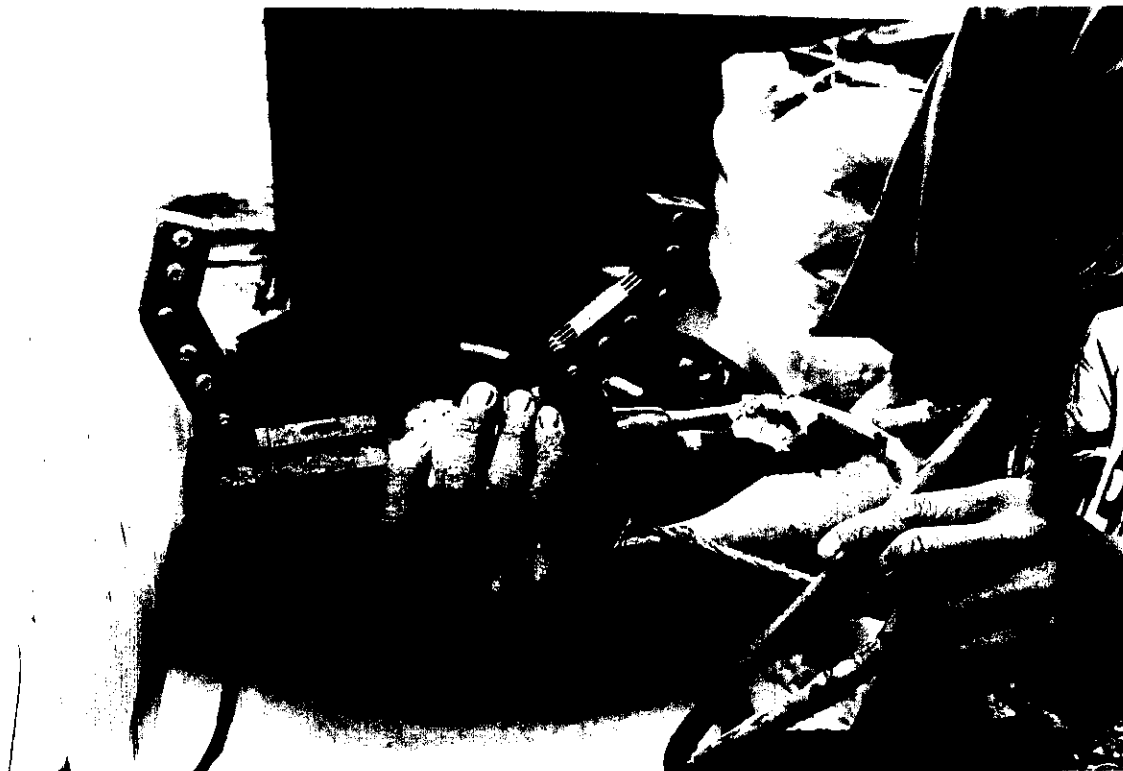




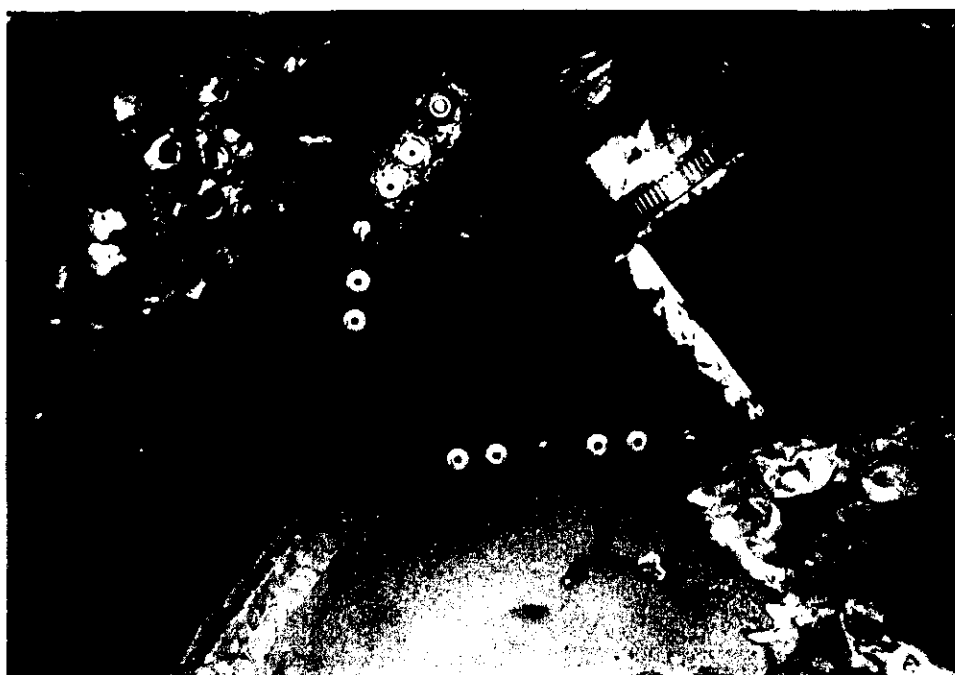
**Photo 2-2 Platform ST 130 Q**



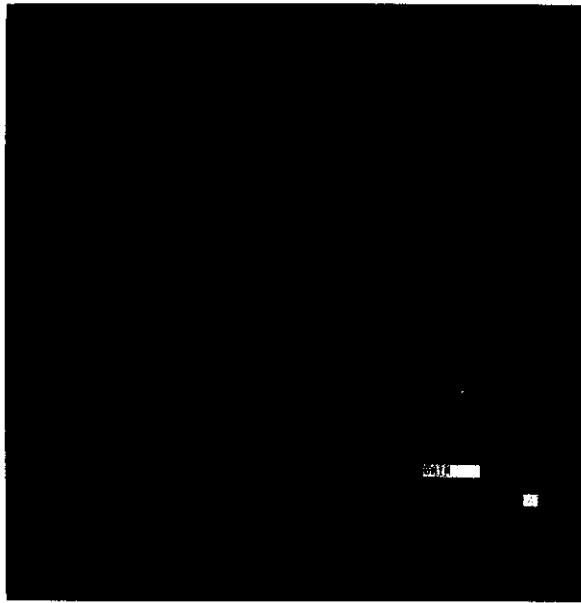
**Photo 2-3 Flooded Member Checking Device**



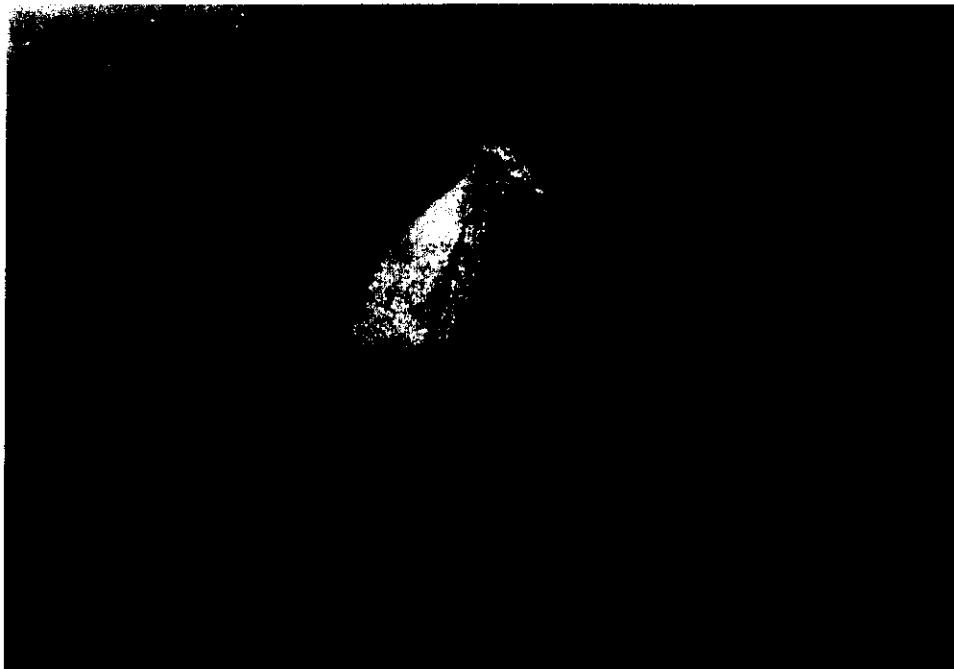
**Photo 2-4 Magnetic Particle Inspection (MPI) Device**



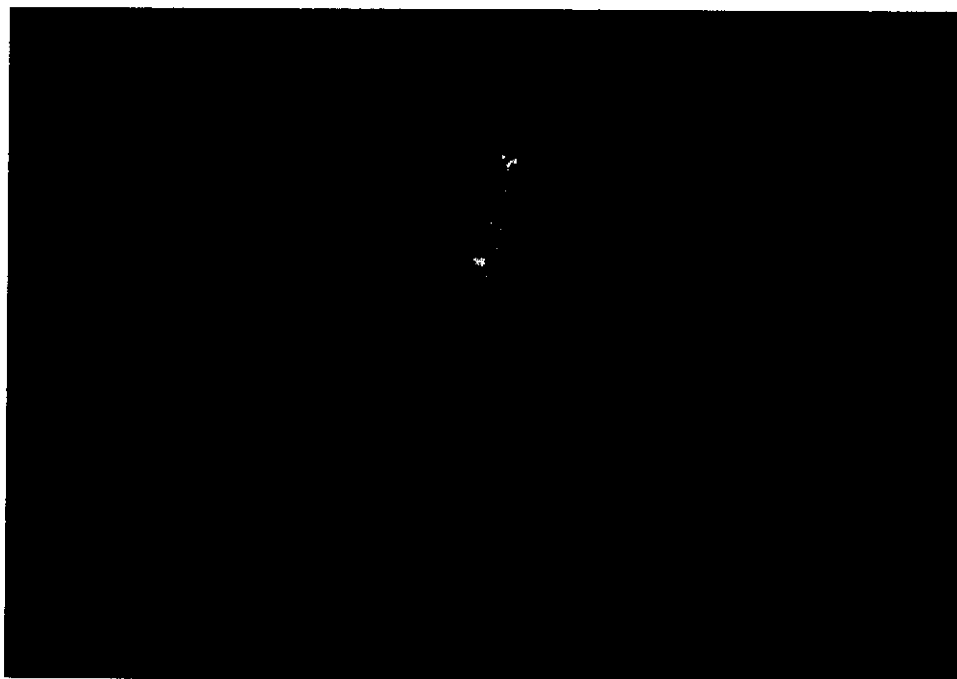
**Photo 2-5 MPI Device at Work on a Joint**



**Photo 2-6 Mesotech Scan of Toppled ST 130 A Platform**



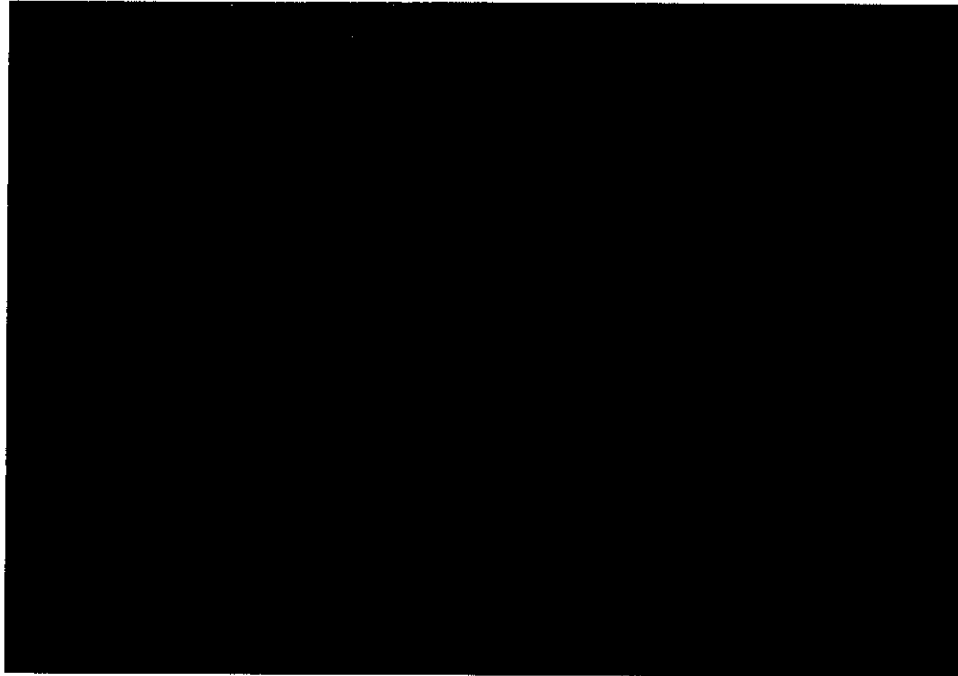
**Photo 2-7 Bending Failure of ST 130 A Leg A1**



**Photo 2-8 Bending Failure of ST 130 A Leg B1**



**Photo 2-9 Bending Failure of ST 130 A Leg C1**



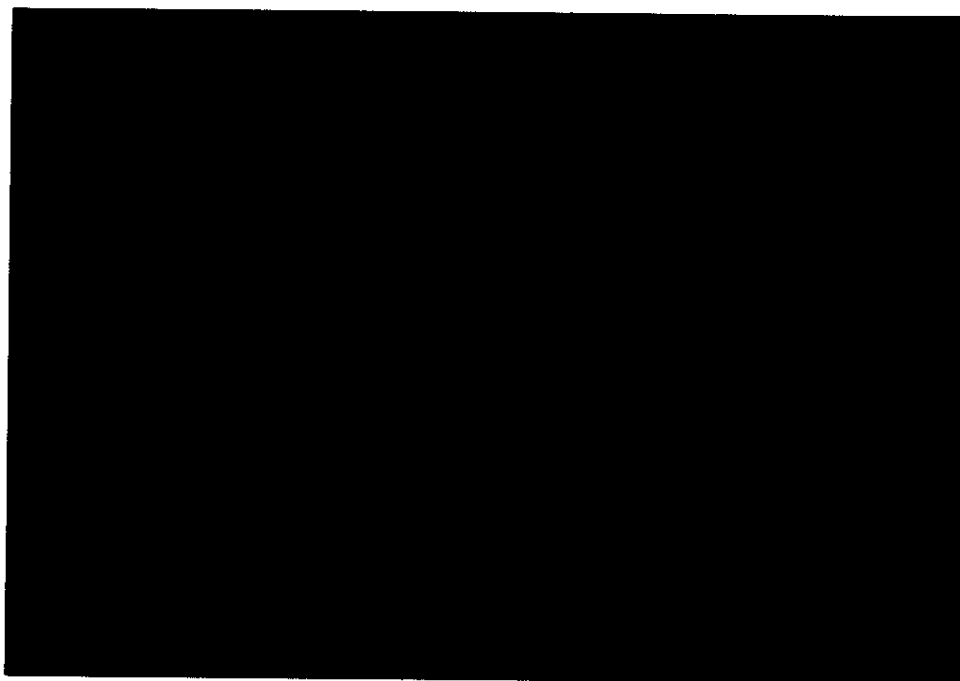
**Photo 2-10 Bending Failure of ST 130 A Leg D1**



**Photo 2-11 ST 151 K - K Joint**



**Photo 2-12 ST 151 K - KT Joint**



**Photo 2-13 ST 151 K - Steel Bar Used to Check K Joint**



**Photo 2-14 ST 151 K - Node at Leg C2 -63' - No Damage**



**Photo 2-15 ST 151 K - Weld at VD A2 -134' - No Damage**





**Photo 2-16 ST 151 K - Weld at Hor. B1 -63' - Missing Material**



**Photo 2-17 ST 130 Q - X Joint**



**Photo 2-18 ST 130 Q - Upper K Joint**



**Photo 2-19 ST 130 Q - VD A1/A2 -130' - Crack**

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10/10/10

## **SECTION 3            PLATFORM ANALYSIS**

### **3.1 Introduction**

This section describes the general approach used for nonlinear structural analysis of each of the three platforms. Specific details of the computer models, analysis cases and analysis results are described in a separate section for each of the platforms.

The computer models used for the analysis were upgraded from those originally used on the Andrew JIP. Upgrades include revisions to the platform models based upon field observations (i.e. removal of boat landings no longer on the platform) and improvements to computer modeling techniques, particularly for joints and the foundation.

The platform models were then subjected to the maximum hindcast Andrew wave to see if the analytical damage matched the observed. The models were then varied to determine if the analytical results could be matched to the observed results. Conclusions were made regarding the capability of the analysis approach in predicting platform behavior.

### **3.2    Computer Models**

PMB's nonlinear computer code Capacity Analysis Program (CAP) was used to perform all of the structural analysis. Computer models for each of the platforms were initially based upon those available from the Andrew JIP, for which the MMS was a participant. The models were updated to reflect the following:

- Actual field configuration. Notes and photos were taken of the platforms during the offshore inspections. These were then used to update the JIP computer models, where appropriate, to reflect actual field conditions. For example, the actual number and location of boat landings (versus what is shown on the drawings).
- JIP findings. The JIP recommended several areas for improvement in nonlinear computer modeling, particularly joint capacities and joint force-deformation characteristics. As explained below, the joint modeling for all of the platforms was improved significantly from that used in the JIP. Estimates of foundation capacity was also an area indicated by the JIP as requiring further study.
- CAP revisions. The CAP computer code has been updated since the JIP with several new features such as new elastic-brittle elements to better mimic failure of K joints. The computer models were updated accordingly to reflect these CAP improvements.

## MMS Platform Inspections and Analysis

Specific revisions to each of the computer models are noted in the appropriate section describing the analysis for that platform.

### **3.3 Environmental Conditions**

Andrew conditions for the specific platform site were based upon those available from the original Oceanweather Hindcast (Oceanweather, 1992), which is the same as that used in the JIP. The JIP evaluated each structure for several directions to account for the hour-to-hour change in hydrodynamic loading on the platform. The JIP concluded that the platform probability of failure was generally controlled by the hour with the largest waves. Therefore, for this project, the only direction evaluated was that for the largest waves acting on the platform. This was felt to be adequate for this project since this study is only trying to compare observed versus actual damaged for each platform, whereas the JIP determined the probability of failure for the platform over several hours of storm loading.

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## SECTION 4 ST 130 A ANALYSIS

### 4.1 INTRODUCTION

A CAP model was built of the ST-130A structure, based on the drawings provided by Chevron, which included drawings of the new deck. The platform was assumed to have four conductors, each 24" diameter by 3/4" thickness, near the center of the middle bay. Pushover analyses were performed using environmental loads during the most severe hour of Andrew. A set of analyses of increasing sophistication were performed and their results compared in order to determine the effect of analysis assumptions. Also, it is thought that the grout between the legs and piles in the second bay from the bottom may have been missing or defective, so analyses were performed in which these leg sections were weakened to determine the effect on platform pushover behavior.

The following analyses were performed:

- Basic Model

Soil strength per API RP 2A, full leg/pile section, elasto-plastic joint failure.

- Increased Foundation Strength Model

Same as the basic model, except soil strength is doubled (increased by 100%), based upon preliminary results of the API/MMS Foundations Project (PMB, 1994) and upon results of the MMS study of the ST 177B Platform (PMB, 1994). This represents a more reasonable estimate of the soil's true strength. These soil properties are used for all subsequent analyses.

- Weak Leg Model

Same as the increased foundation strength model, except the leg/pile section in the second bay from the bottom is replaced with the properties of the leg only (for all legs).

- Brittle K-Brace Model

Same as the increased foundation strength model, except K-braces are modeled to fail in a brittle mode. Similar K-joints in the ST 177 B platform were observed to fail in a brittle mode (PMB, 1994d). However, this model does not represent true brittle failure of the chord itself, since it is the brace that is modeled as brittle and not the joint.

- Brittle K-Brace, Weak Leg Model

Same as brittle brace model, except with leg properties from the weak leg model.

- Brittle K-Joint Model

Same as double foundation model, except not only the K-braces but also the chord of the K-joint is modeled to fail in a brittle fashion.

- Brittle K-Joint, Weak Leg Model

Same as brittle joint model, except with leg properties from the weak leg model. This is believed to be the most accurate modeling of the pre-Andrew platform.

In addition, a pair of preliminary analyses were performed with the platform completely fixed at the mudline (i.e. no pile failure possible), for the full leg/pile case and for the weak leg case, to determine where on the legs the first leg failure would occur for each case.

## **4.2 DESCRIPTION OF MODEL**

A plot of the model is shown in Figure 4-1.

### **4.2.1 Substructure**

The substructure was modeled explicitly per the drawings, with the following exceptions and comments:

- Node cans were neglected (except for estimating joint capacities).
- The secondary conductor framing (at three levels) was not included explicitly in the model. The weight in water for the conductor framing was calculated and added to the model as lumped weights at the closest model nodes.
- The hydrodynamic properties of the boat bumper were calculated and added to the model using a stick element at the water line.
- Risers and caissons were neglected.



### **4.2.2 Superstructure**

The lower portion of the superstructure, from the bottom of the lowest deck to the base of the superstructure, was modeled explicitly, similar to the substructure. The decks were modeled approximately, with effective member sizes chosen to provide the estimated values of the deck stiffness and weight. The deck plates were represented by stiff horizontal X-braces.

A deck equipment weight of 1400 kips was used. It was spread evenly among all three decks according to tributary area.

### **4.2.3 Conductors**

The four conductors were lumped together in one effective element string in the center of the middle bay. The conductor below mudline was replaced with a linearized element representing the estimated stiffness of the conductor and its foundation. The PMB pile analysis program PAR was used to analyze its behavior and obtain its stiffness matrix, as previously determined in the Andrew JIP.

### **4.2.4 Piles and Soil**

Full pile make-up information was not available, so the pile make-up from the similar ST 151 K structure was used. The pile penetration is 175 ft. The pile diameter is 30".

Soil properties were also taken from ST 151 K.

### **4.2.5 Element Types**

The following element types were used for the various portions of the basic model:

- Legs and piles: yielding beam-column elements, with plastic failure behavior appropriate for A36 steel (i.e. the stiffness softens for a while before fully plastic behavior is reached)
- Longitudinal frame diagonal braces: buckling Marshall struts
- Transverse frame K and X braces: nonlinear struts with capacity adjusted to reflect calculated joint capacity; in the earlier models, the elements go fully plastic immediately after yield, while in the later models they experience brittle failure soon after yield
- All members expected to behave linearly (deck, plan horizontals, plan bracing): linear

beam elements

For the analysis models with brittle K-joint capability, the compression side of the two linear chord elements at each K-joint was replaced with a brittle beam-column element that undergoes brittle failure soon after yielding. This is discussed further in Section 4.4.

### **4.3 ENVIRONMENTAL LOADS**

The environmental loads were based on the Hurricane Andrew data for the worst hour of the storm, as follows:

- Maximum wave height = 61 ft
- Associated peak spectral period = 14.43 sec
- Associated current velocity = 2.0 knots = 3.4 ft
- Maximum wind speed = 45 m/sec = 150 ft/sec
- Environmental direction =  $286.6^\circ$  off true north, or  $311.6^\circ$  from platform north, approximately acting diagonally on the platform.

The following environmental assumptions were made, based on API RP 2A, 20th edition:

- Storm tide and surge = 3-1/2 ft
- Current blockage factor = 0.85 (for diagonal direction)
- Current profile is constant for water this shallow
- Wave spreading factor = 0.88
- Conductors are too far apart in the diagonal direction relative to their diameter to provide any shielding
- Marine growth is a constant 1.5" (on radius) for all elements between 1' above mean water line and the mudline in water this shallow
- Drag coefficient = 1.05, inertia coefficient = 1.2 (for rough members, as there is marine growth on all members below water line)

The wave period used in the pushover load generation was 1.04 times the peak spectral period, or 15.0 sec. After applying the Doppler shift due to current (as described by API RP 2A, 20th), the effective wave period is 15.8 sec.

The wave does not reach the deck, so wave forces on the deck were not considered.

Wind load on the deck was calculated based on API RP 2A (20th) recommendations. The total wind force is 86 kips in the transverse (broadside) direction and 45 kips in the longitudinal (end-on) direction.

The combined wave and current load generated by CAP based on the above assumptions is 1984 kips in the transverse direction, 1706 kips in the longitudinal direction, and 86 kips in the downward direction.

Thus the total environmental load used in the pushover analyses was 2070 kips in the transverse X direction and 1752 kips in the longitudinal Y direction. The resultant is 2713 kips. In the pushover analysis results, the "load ratio" is the actual pushover load divided by the resultant load.

#### **4.4 MEMBER AND JOINT CAPACITY CALCULATION AND MODELING**

##### **4.4.1 Member Failure Mode Determination**

Joint capacities for the X- and K-joints were calculated as discussed in the next section. The broadside frame diagonals are not affected by joint capacity as the legs are fully grouted.

For the transverse frame X and K braces, the yielding and buckling capacities for the members were calculated, and it was found that joint capacity controls for these elements. Thus, these braces were modeled using nonlinear truss elements with adjusted yield capacity.

For the broadside frame diagonals, the yielding and buckling capacities were calculated, and it was found that buckling controls. Thus, these braces were modeled using buckling Marshall strut elements, which automatically model the braces' buckling behavior.

##### **4.4.2 Joint Capacities**

Joint capacities were calculated for the X and K joints in the transverse frames. The API RP 2A joint capacity equations were not used because they represent a lower bound, not mean, capacity, and, even with that difference accounted for, they are overly conservative according to later laboratory test results (the API equations are based on tests performed before 1980).

Thus, a more recently-derived set of joint capacity equations were used, as summarized below. See the similar study of the ST 177 B platform (PMB, 1994d) for more details.

The X- and K-joint capacities were calculated using the formulas presented in a pair of papers, OTC 4189 (Billington, Lalani and Tebbett, 1982) and BOSS '88 (Ma and Tebbett), with API's  $Q_f$  term added to account for the effects of chord stress (as these papers do not study the effects of chord stress). Based on the platform drawings and loading direction during Andrew, it was determined that the brace members were the compression portion of the X, and the chord members were the tension portion of the X; thus the axial compression, not tension, formula is applicable. The out-of-plane bending formula was also used, as discussed in the following paragraph. Because the braces are modeled as truss elements, their in-plane bending moment is not modeled; however, this moment component is expected to be small and should not significantly affect the behavior of the brace or structure.

The joint capacities were modeled by adjusting the yield stress of the brace elements so that they fail at the appropriate load level. This effective yield stress was adjusted to account for the effects of chord stress and lateral hydrodynamic forces according to the following methodology:

- (1) Estimate the stress level in the chord in order to calculate a preliminary  $Q_f$  factor for use in the joint capacity equations. Calculate the axial capacity and out-of-plane bending capacity.
- (2) Estimate the maximum water particle velocity (maximum wave-induced plus current with blockage) at the brace's average water depth.
- (3) Given the element length, estimate the brace-end moments due to the drag force on the element. Assume partially fixed ends (i.e. the moment is 1/10 of the distributed load time the length squared). Assume a drag coefficient of 1.05 (for rough members with marine growth). For X braces, take the full length of each brace, not the half-length, because the crossing brace does not provide much restraint against out-of-plane bending.
- (4) Use the API RP 2A (20th) axial/bending interaction equation. For simplicity, assume the moment calculated in the previous step is out-of-plane. Find the axial force at combined axial/bending failure with this moment, based on the axial and out-of-plane bending capacities calculated in the first step. Take this to be the effective axial capacity for the element.
- (5) After the analysis, determine the actual stresses in the chord at the end of the load step when the adjacent brace fails. Each of the three chord stresses (axial, in-plane bending, out-of-plane bending) should be calculated as the average of the stresses in the two

adjacent chord elements, accounting for sign. If the combined (SRSS) stress is not close to the one originally estimated in the first step, go back to the first step with these improved chord stresses and repeat.

For the base case analysis, it was found that the chord stresses converged in one iteration (in addition to the original analysis). A second iteration was performed to verify convergence. For the next several analysis cases, the chord stresses at failure were checked, and they were found to be close to the ones found in the base case. Thus, for the remainder of the analyses, the same joint capacities were used.

### **4.4.3 Brittle Brace Modeling**

Based on empirical evidence, K-braces of the sort used in this structure fail in a brittle fashion, i.e. the joint fails completely and cannot carry any load. Thus, in some analyses, the K-braces were replaced with brittle truss elements, which fail completely and cease to carry load after they reach their effective yield stress.

### **4.4.4 Brittle Joint Modeling**

The K-joint failures observed in the similar ST 177 B structure involves the entire joint, not just the braces; when the joint failure occurs, the chord shears completely through between the tension and compression braces. For the analysis cases which model this brittle failure of the entire K-joint, another iterative solution was used. One of the two adjacent chord elements was replaced with a yielding beam-column element, which would experience a complete brittle failure soon after yielding. As shown in Figure 4-2, this effectively destroys the load path through the joint.

First, an analysis was performed in which each chord beam-column element was given an arbitrarily high yield stress to preclude it from failing. The actual axial force and bending moments at the ends of each beam-column chord, at the end of the load step when the adjacent brace fails, were obtained. Based on these forces, an effective chord yield stress was calculated which would cause the chord to yield by the end of the load step in which the brace fails. The analysis was then rerun with this yield stress in the chord. (The tension K-brace was given a large yield stress, as it does not need to fail in the model to get correct model behavior, as can be seen in Figure 4-2.)

## 4.5 ANALYSIS RESULTS

### 4.5.1 General

A summary of capacities for the various analyses, in the form of pushover load ratio at failure, is presented in Table 4-1.

A summary plot of the pushover load ratio as a function of deck X-direction lateral displacement for all major analyses is shown in Figure 4-3. In this figure, the following abbreviations are used to identify the analyses: DF = double foundation strength, WL = weak leg section, BB = brittle K-braces (but not joints), BJ = brittle K-joints. These results will be discussed in the following sections.

Analysis results presented in the following sections include schematics of the platform computer model taken from CAP that indicate which model elements have experienced nonlinearity, i.e. yielding or other failure. For beam-column elements, a small dark "blot" at one end of the element indicates that first yield has been reached at that end. Further softening events, which approximate fully plastic behavior, are indicated by larger-diameter "blots". (In analysis cases with brittle failure of beam-column elements, these elements fail very soon after first yield. After brittle failure, the relevant member ends are marked with the larger-sized "blots".)

Analysis results also include plots of pushover load ratio versus top deck displacement in the transverse X lateral direction. Because the environmental direction is roughly diagonal, there is also longitudinal Y-direction lateral displacement, which is not included in the results presented. The X-direction is the weaker of the platform's two lateral directions. It produces the more interesting behavior, and thus is the direction used in the plots. Displaced shape plots will also be presented so as to show the X-direction behavior. For most of the displaced shape plots, the displacements have been exaggerated 20 times.

Unless otherwise noted, the term "failure" indicates nearly fully plastic behavior. Nonlinear element behavior which has not yet become nearly fully plastic is called "yielding". "Brittle failure" means that the element in question has shed all load.

### 4.5.2 Basic Model

The capacity of the basic model is 1.20 times the Hurricane Andrew environmental pushover load. The yielded and failed elements at the time of the last load step are indicated in the plot in Figure 4-4. This analysis terminated due to large displacements before all the piles had become plastic, but it is clear from the detailed results that the platform's global failure would be caused by lateral pile failure. At this time, no platform legs have yielded, though nearly all

the transverse K-braces have failed (dashed lines in Figure 4-4).

The pushover load ratio plot is shown in Figure 4-5, with major events indicated. The general behavior is that of a gradually, smoothly, decreasing stiffness, with no abrupt changes in behavior until final failure. The first elements to fail are the bottom K-braces, which begin failing at 0.525 times the load ratio.

Sample displaced shape plots are shown in Figures 4-6 and 4-7. Figure 4-6 occurs at a load ratio of 1.1. Note that the lowest bay has experienced far more shear deformation than the rest of the platform, as all K-braces in this bay have failed by this time. Note also that a significant portion of the deck displacement is due to pile deformation. Figure 4-7 shows the structure even later, at a load ratio of 1.175 (shortly before global failure). In this plot, the deformation of the piles is even more pronounced.

### **4.5.3 Increased Foundation Model**

The capacity of the increased foundation strength model is 1.55 times the Hurricane Andrew environmental pushover load, or about 30% greater than the basic model's capacity. The yielded and failed elements at the time of the last load step are indicated in the plot in Figure 4-8. The failed elements are similar to those in the basic model except that a few more braces have failed before pile failure. In this case, the analysis continued until all piles failed in laterally due to bending, similar to the base case model but at a higher total lateral load acting on the platform.

The pushover load ratio plot is shown in Figure 4-9, with major events indicated. The behavior is similar to that of the base case, except that, as expected, it is slightly stiffer for equivalent load ratios and global failure occurs later. The first elements to fail are still the bottom K-braces, at virtually the same load ratio as before.

For this analysis case, a series of displaced shape plots are presented in Figures 4-10 through 4-14, to illustrate the changes in the platform's behavior with increasing load. In Figure 4-10, the platform's displacement is virtually uniform when the first braces begin to fail. In Figure 4-11, which represents the very next load step, the platform's bottom bay becomes more deformed than the rest of the structure because of the failure of the bottom bay K-braces. Figure 4-12 shows this behavior continuing. By the time of Figure 4-13, other parts of the platform have undergone large deformations because of the failure of the rest of the K-braces. Figure 4-14 shows the shape at the last load step. Note that the platform is undergoing double curvature, i.e. the upper portions of the platform are curving back towards the loading direction. A similar double curvature was noted in the onshore inspection for the ST 177 B platform (PMB, 1994d). Also note that the platform deformations are larger relative to the pile

deformations than in the base case, because the soil is stronger.

Note that the increased foundation strength modeling is used in all subsequent analyses, as it is thought to more adequately represent realistic soil behavior.

### **4.5.4 Fixed Base Models**

In order to compare where the legs will first fail for the two leg section cases, analyses were performed with the foundation completely fixed at and below the level of the mudline. The full leg/pile case fails in the second bay from the top and in the bottom bay (near the mudline) at virtually the same load level, before the second bay from the bottom has yielded at all (where the platform was found to have failed per the inspection), as shown in Figure 4-15. As expected and as described below,, the weak leg case failed in the bay with the weak leg (second from the bottom). Thus, leg failure is expected to occur in the second bay up from the bottom (as in the real structure) only if the leg/pile section is unexpectedly weaker there than in the adjacent two bays.

### **4.5.5 Weak Leg Model**

The capacity of the weak leg section model is 1.50 times the Andrew environmental load (worst storm hour). This is slightly lower than the previous analysis' load ratio. The yielded and failed elements at the time of the last load step are indicated in the plot in Figure 16. In this case, there is yielding in all of the legs in the weak bay, with some of the legs exhibiting plastic behavior. However, it is still pile failure that controls the platform capacity.

The pushover load ratio plot is shown in Figure 4-17, with major events indicated. It is similar to the previous analysis, except that it is slightly less stiff (as expected) once the weak legs yield. However, the difference is not large. The displaced shape plots are not substantially different from those of the increased foundation analysis, except for slight effects of the additional flexibility in the weak legs.

### **4.5.6 Brittle K-Brace Model**

The capacity of the brittle K-brace model is 0.76 times the Hurricane Andrew environmental pushover load. This is the first modeling case with a pushover load ratio of less than unity, suggesting that failure of this structure could have been predicted using this type of model.

The yielded and failed elements at the time of the last load step are indicated in the plot in Figure 4-18. This shows that the legs in the second bay down from the top experience initial yielding. However, the piles are still the first elements to become fully plastic and hence cause



the global failure.

The pushover load ratio plot is shown in Figure 4-19, with major events indicated. The bottom K-braces fail in a brittle fashion, which causes a temporary plateau in the load ratio. This is in contrast to the smooth curves seen in the previous analyses. The load level then increases until all other failures occur in rapid succession, within a load ratio difference of 0.0125. (The Y-direction lateral displacement does not show any such plateau until final global failure, as the braces in those frames are intact until near the ultimate capacity.)

### **4.5.7 Brittle K-Brace, Weak Leg Model**

The capacity of the brittle K-brace, weak leg, model is 0.80 times the Hurricane Andrew environmental pushover load. As seen in the non-brittle models, the weak leg makes little difference to the overall platform capacity. In this case, it even increased the capacity very slightly, probably due to load step size and numerical stability during the analysis.

The yielded and failed elements at the time of the last load step are indicated in the plot in Figure 4-20. This model remained stable slightly longer than the previous version with the full leg/pile section, so a few additional elements failed before global failure compared with the previous one. Though yielding occurs in the weak legs, the piles still control the global capacity. This is consistent with the results for the previous full leg/pile and with weak leg models without brittle failure.

The pushover load ratio plot is shown in Figure 4-21, with major events indicated. It is very similar to that for the previous full leg/pile analysis case.

The displaced shape at the next-to-last load step is shown in Figure 4-22. The deformation of the bottom bay is even more pronounced compared with the rest of the structure than in the previous non-brittle analyses, because the stiffness of the bottom bay is reduced when the K-braces shed all their load. Note that physical examination of the failed platform also indicated complete failure of the k joints and legs in this region, as described in Section 2.2.

### **4.5.8 Brittle K-Joint Model**

The capacity of the brittle K-joint model is 0.71 times the Hurricane Andrew environmental pushover load. Brittle failure of the chords of the K-joints thus decreases the platform's capacity by about 7%. As in all previous analyses, the bottom K-joints were the first part of the structure to fail, as shown in Figure 4-23. This figure shows the failed components at a pushover load ratio of 0.55, which, as in all previous analyses, is when the K-joints failed. All of the failed elements in this plot have failed in a brittle fashion.

The yielded and failed elements at the time of the last load step are indicated in the plot in Figure 4-24. All the K-joints have failed, several other braces or joints have failed, and the legs in the second bay from the top have yielded. However, as in all previous analyses (except fixed base), global failure was caused by the piles.

The pushover load ratio plot is shown in Figure 4-25, with major events indicated. It is similar to the plot for the previous brittle brace (but not joint) analysis, except that the region of load increase between the bottom bay K-joint failure and the global failure is slightly shorter.

For comparison, the pushover load ratio plot for Y-direction (longitudinal) lateral displacement is shown in Figure 4-26. Because no braces in the longitudinal frames fail until global platform failure, the load/displacement behavior is linear until the very last step.

The displaced shapes of the platform for several steps in the analysis are shown in Figures 4-27 through 4-30. Figure 4-27 shows the platform just before the bottom bay K-braces all fail. Figure 4-28, shows the platform in the next load step, after these braces fail. Note that at this time, the platform's deformation becomes concentrated in the bottom bay because of its greatly reduced stiffness after the braces fail. Figure 4-29 shows the platform at the next to last load step; its behavior is the same as in Figure 4-28, except the magnitude of the displacements have increased. Figure 4-30 shows the platform's shape at global failure. Note the large deformations along much of the length of the platform, because the braces have failed in all bays of the transverse frames. Note that after the bottom bay K-brace failures, the upper portion of the platform leans back in the direction of the load (again as seen for ST 177 B).

### **4.5.9 Brittle K-Joint, Weak Leg Model**

The capacity of the brittle K-joint, weak leg, model is 0.70 times the Hurricane Andrew environmental pushover load. This is slightly lower than that of the previous analysis case, so the weak leg has little effect on the global platform capacity (which was also seen in previous analysis cases). However, it dramatically changes the failure mode, as described below.

As before, the bottom K-joints fail first, at a load ratio of 0.55. The weak leg section yields soon thereafter. The yielded and failed elements at the time of the last converged load step are indicated in the plot in Figure 4-31. In this case, the global failure is caused not by the piles, but by the weak leg sections. Although the load step following this one did not converge, the results indicated large unbalanced loads associated with the area containing the bottom bay K-joints and the weak leg joints at the same elevation. This indicated that failure would likely occur in this region, that is, at or just above plan elevation -105'. At this time, no members in the higher bays have yielded or failed.

The pushover load ratio plot is shown in Figure 4-32, with major events indicated. The general behavior is the same as that of the previous full leg/pile analysis, but the second up-slope region is less stiff (because of the yielded legs) and the global capacity is slightly lower than previous analyses.

The predicted response of the platform using this computer model is remarkably similar to that of the toppled platform as observed in the underwater inspections described in Section 2. The platform toppled in essentially one piece with little or no damage to the upper X or K joints. The structure broke at approximately the second bay from the mudline with severe damage to the K joints and to the legs. There was no foundation damage as could be discerned by the divers.

This leads to the possible explanation for the failure of this platform in this manner due to the lack of grout in the leg annulus at approximately the -100 elevation. This lack of grout is most likely due to an inadequate grout job during installation. It is not likely due to any lack of care or maintenance by the operator during the many years of operation of the platform.

### **4.6 CONCLUSIONS**

The actual failure of the structure involved complete breaking of the structure partway up the second bay up from the bottom. There was no grout observable in the broken leg/pile section during the MMS platform inspection, which would act to make that section weaker than expected. There was no observable foundation failure.

The earlier, less sophisticated, analyses cases predicted a global platform capacity driven by pile failure at approximately one and a half times the Andrew environmental pushover load. Clearly, this is not consistent with the observed behavior.

When the brittle failure of the transverse frame's K-joints is properly modeled, however, the structure fails at less than the Andrew pushover load. This explains the platform's observed failure.

While the issue of leg/pile strength in the second bay up from the bottom does not significantly affect global platform capacity, it does change the predicted failure mode. In all analyses except the last one with fully brittle K-joints and weak leg/pile section, global platform failure was caused by pile failure. This is inconsistent with the observed behavior.

However, in the last analysis case, which is the estimate of the computer model most accurately reflecting the actual physical structure, global failure was caused by failure of the platform in the second bay up from the bottom. This matches the observed behavior very closely. The

## MMS Platform Inspections and Analysis

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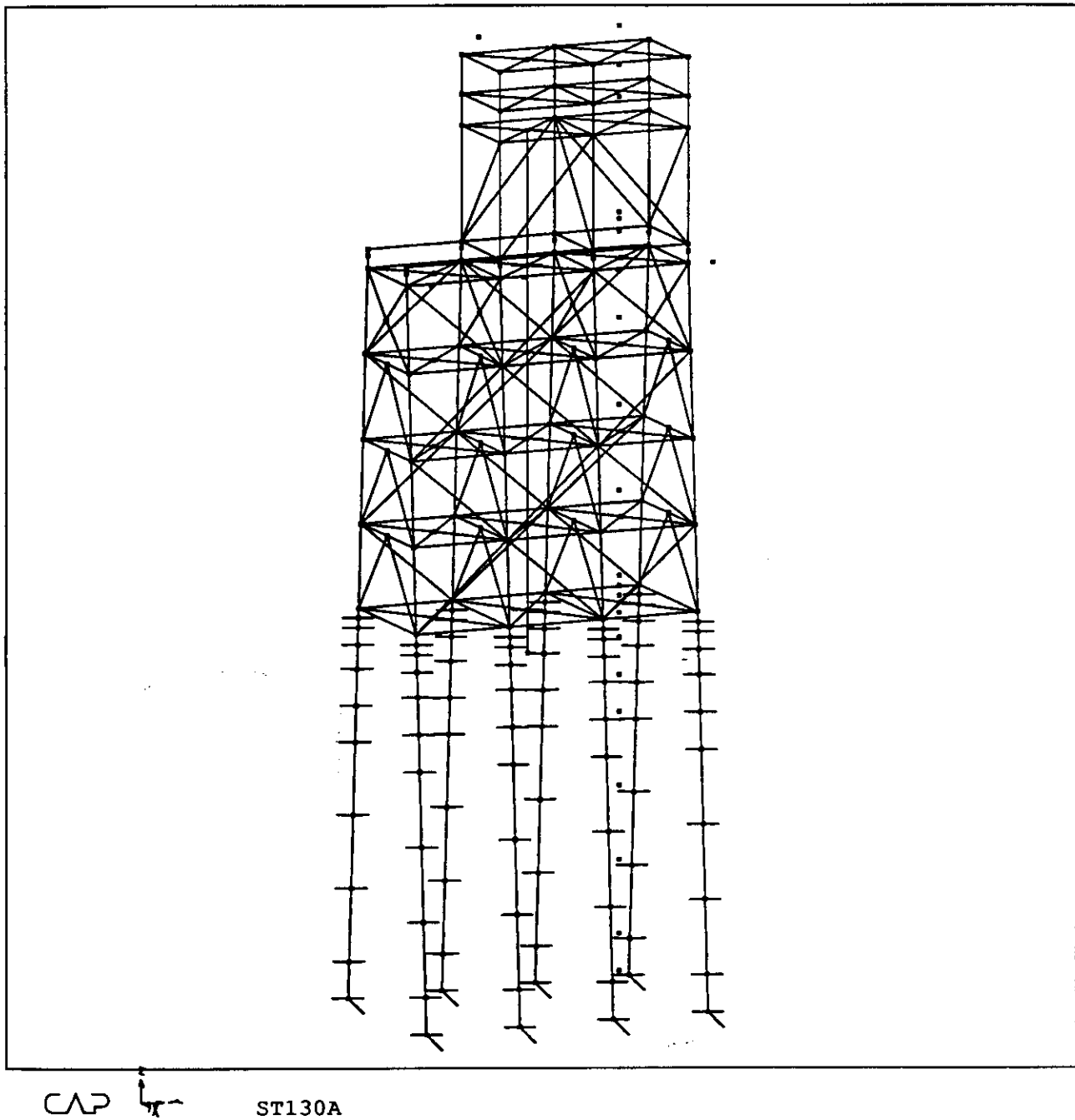
failure in the model occurred just above plan elevation -105', while the failure in the physical structure occurred further up the same bay. This is explainable if the grout was defective for only part of the bay, that is if the leg/pile section was at full strength just above the -105' elevation plan and grout failure started somewhere higher.

It is anticipated that the last two analysis cases, with brittle K-joints with and without the weakened leg section, bracket the best estimate of the platform's behavior. In the first case, the full leg/pile section was used in the "weak" bay. To represent the effects of grout failure in the second of these models, the pile was completely neglected in determining the stiffness and strength of the weak section. In reality, the pile is still there (although its load connection to the structure is more tenuous), and it will contribute at least some of its strength and stiffness. Thus, it is expected that the real behavior is somewhere between these two extremes.

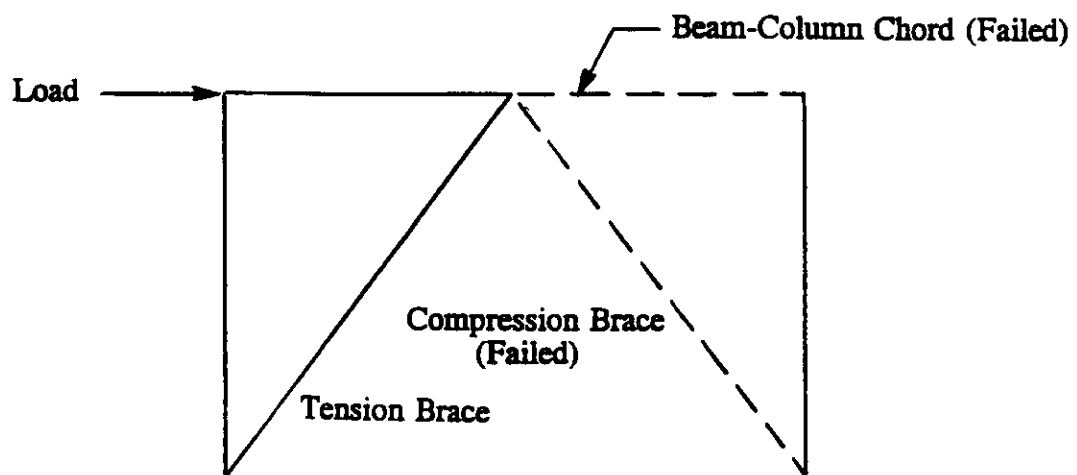
Analysis Case	Pushover Load Ratio at Failure
Basic Model	1.20
Double Foundation Model	1.55
Weak Leg Model	1.50
Brittle K-Brace Model	0.76
Brittle K-Brace, Weak Leg Model	0.80
Brittle K-Joint Model	0.71
Brittle K-Joint, Weak Leg Model	0.70

Note: Other than the basic model, all models have doubled foundation strength.

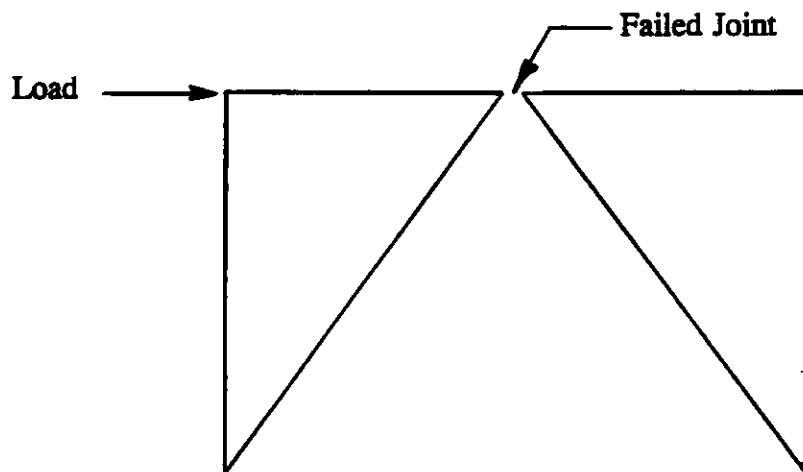
**Table 4-1 Summary of ST 130 A Results**



**Figure 4-1 ST 130 A CAP Computer Model**



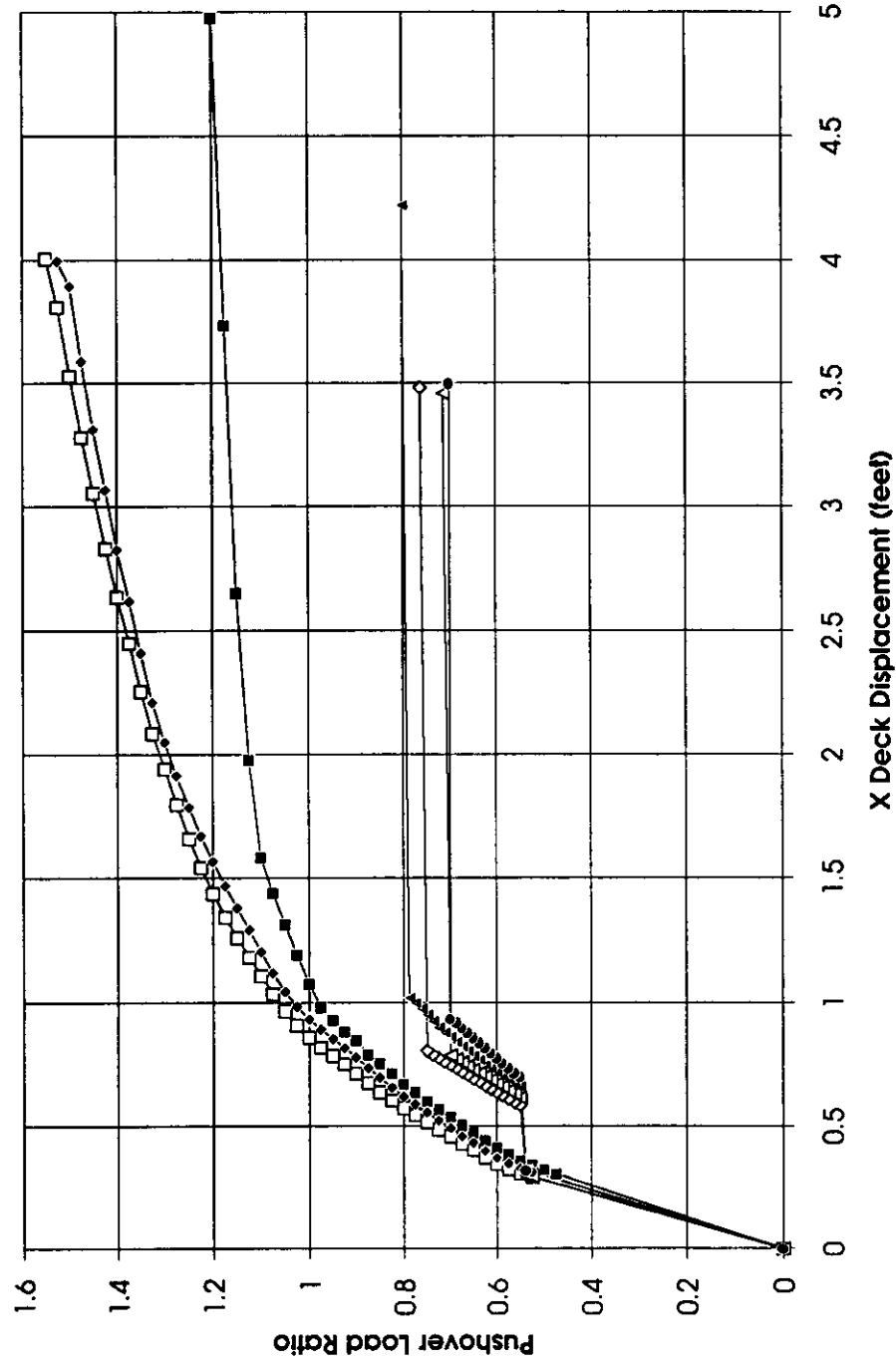
MODEL



ACTUAL STRUCTURE

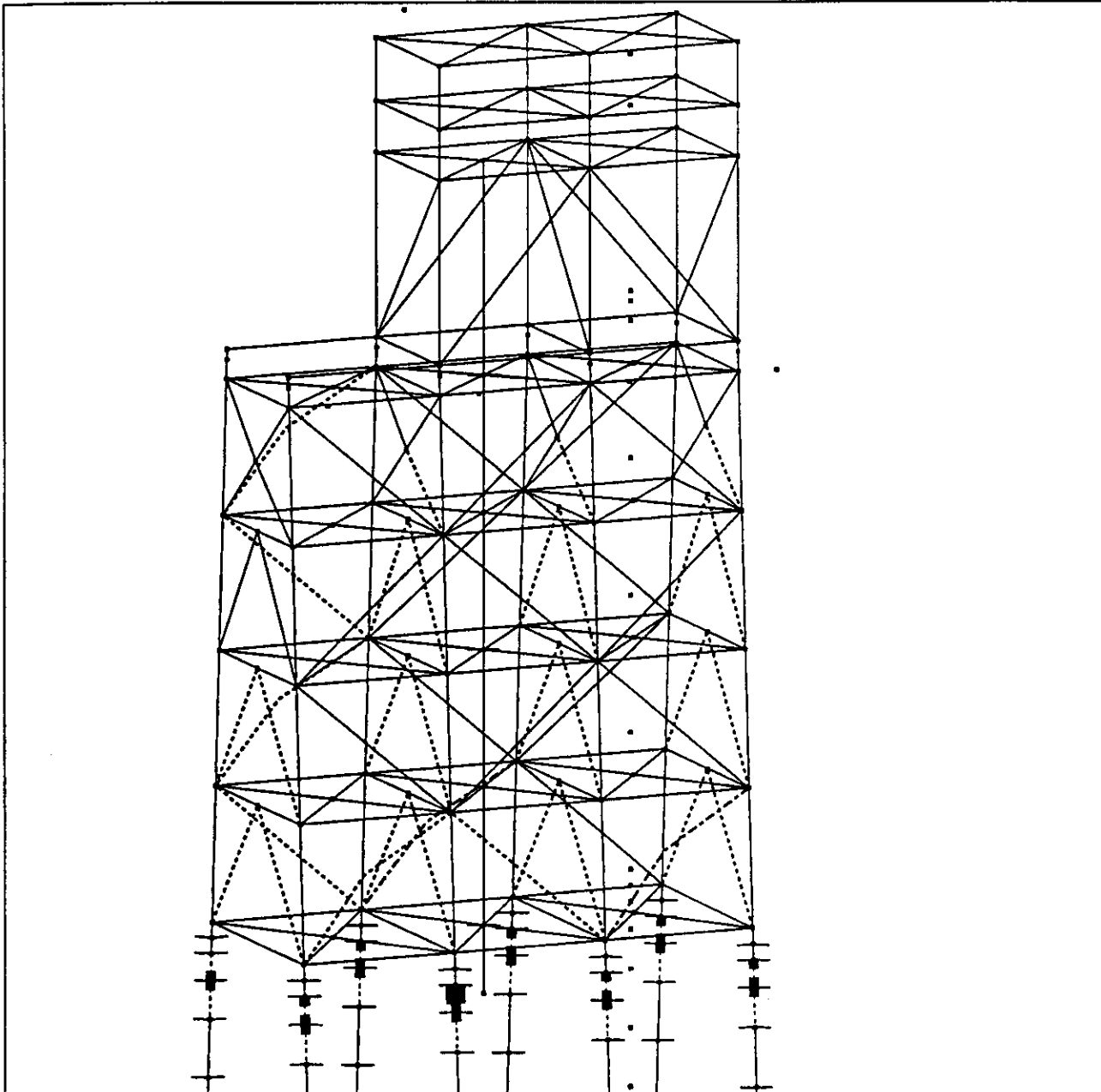
**Figure 4-2 K-Brace Joint Failure**

**ST130A: Results Comparison for Various Modeling Methods**



**Figure 4-3 ST 130 A Results Comparison**





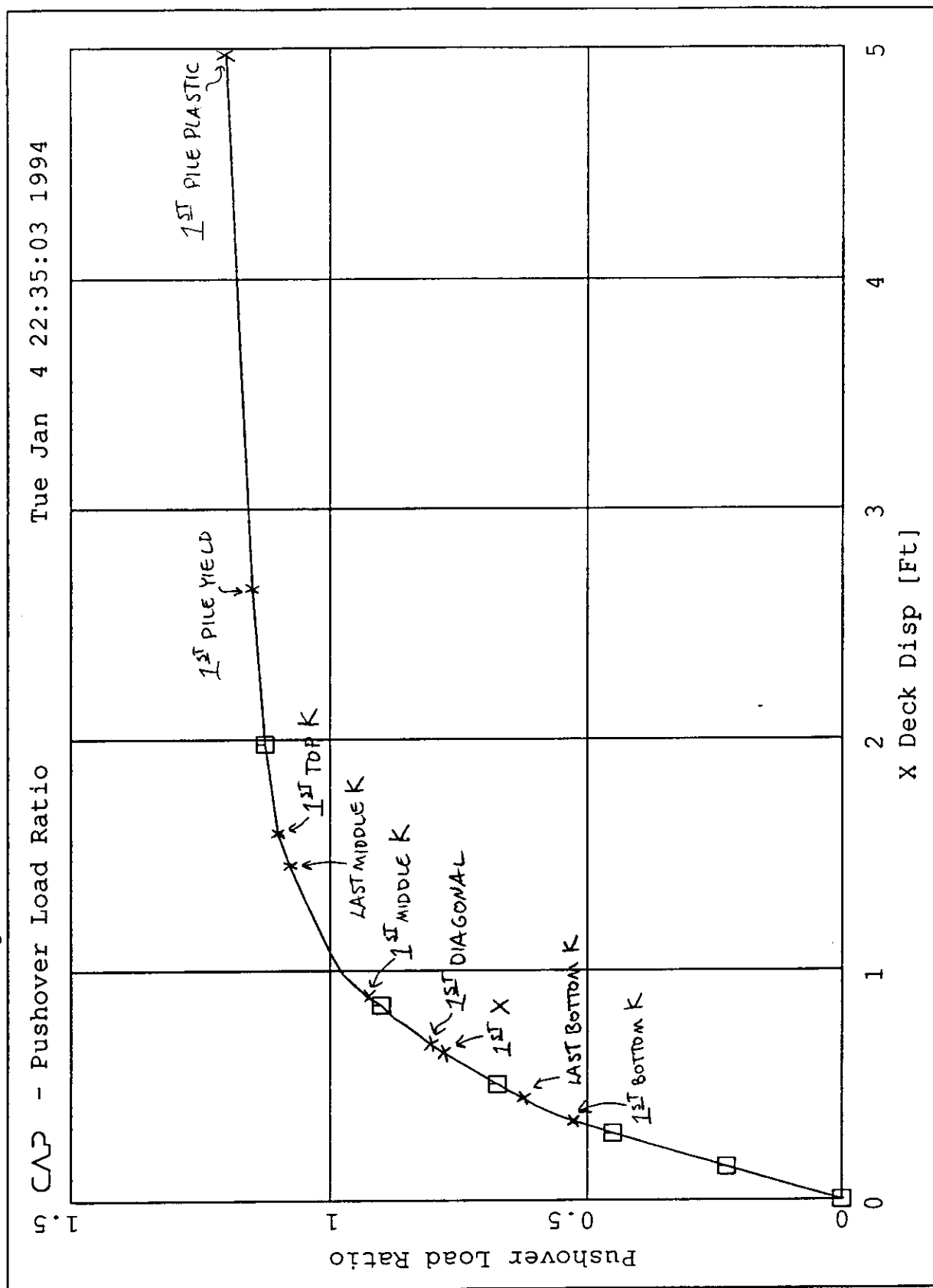
CAP 4-

Basic Model: Failed Components

Inelastic Events Legend

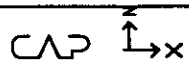
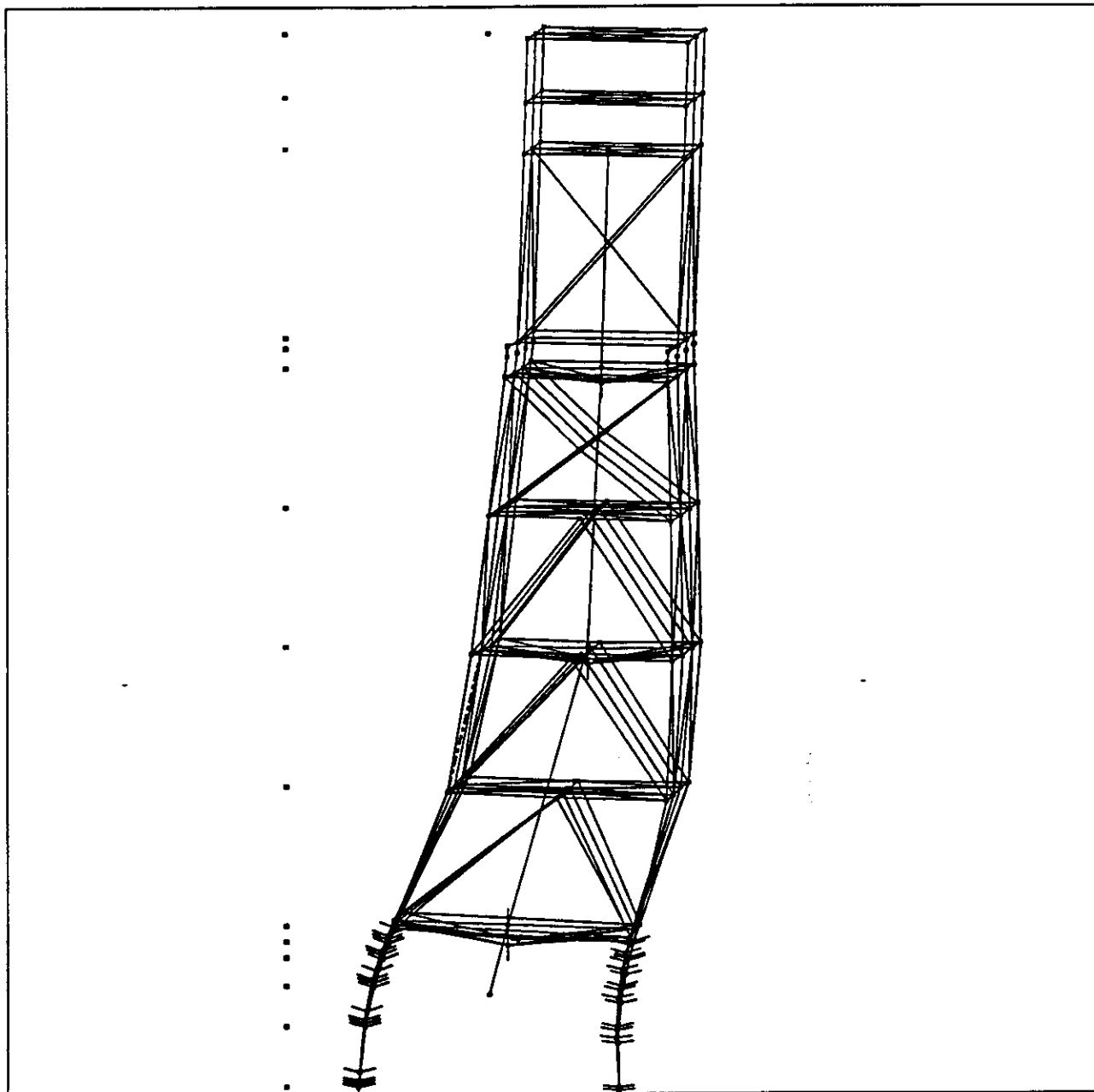
————	Elastic	.....	Strut Unloading
.....	Strut Residual	-----	Strut T/C Post Buckling
.....	Plastic Strut/NLTruss	.....	Beam Clmn Initial Yield
.....	Beam Clmn Fully Plastic		

**Figure 4-4 Basic Model Failed Components**



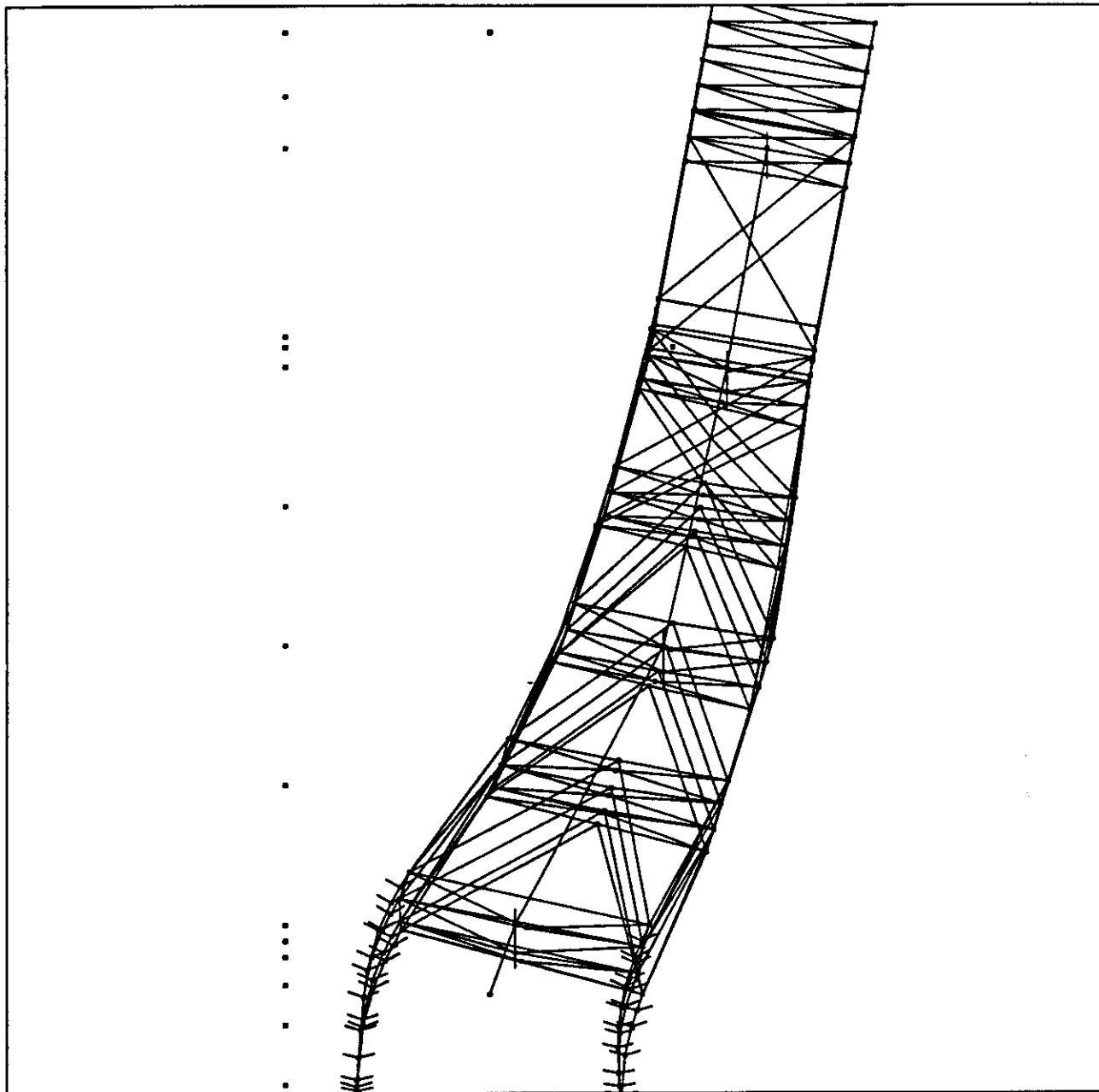
Basic Model (iter 2)

**Figure 4-5 Basic Model Results**



Basic Model: Step 45

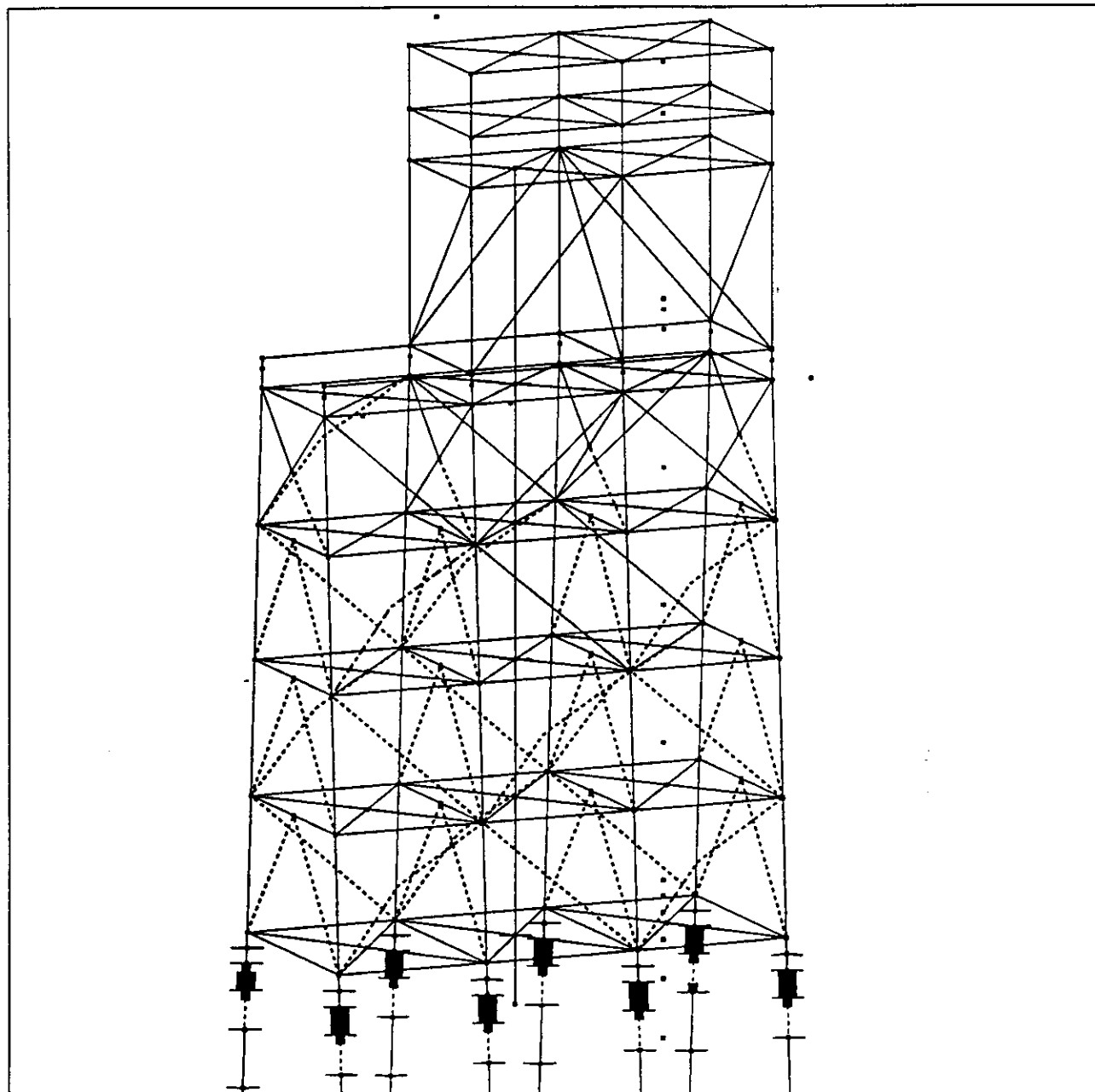
**Figure 4-6 Basic Model: Load Ratio = 1.100**



CAP  $\uparrow$  x

Basic Model: Step 48

**Figure 4-7 Basic Model: Load Ratio = 1.175**



CAP

Double Foundation Model: Failed Components

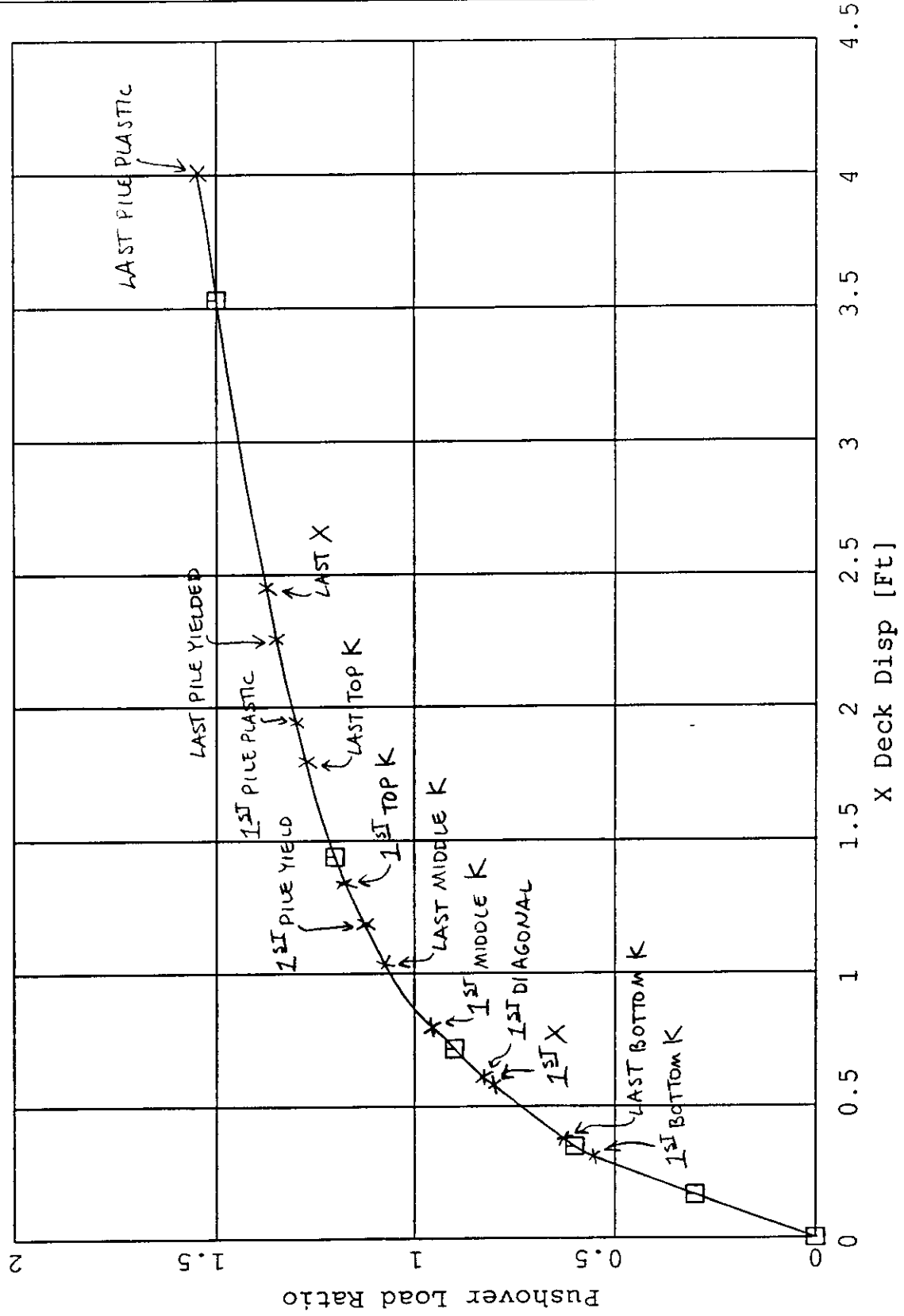
Inelastic Events Legend

————	Elastic	.....	Strut Unloading
.....	Strut Residual	.....	Strut T/C Post Buckling
.....	Plastic Strut/NLTruss	.....	Beam Clmn Initial Yield
.....	Beam Clmn Fully Plastic		

**Figure 4-8 Increased Foundation Strength Failed Components**

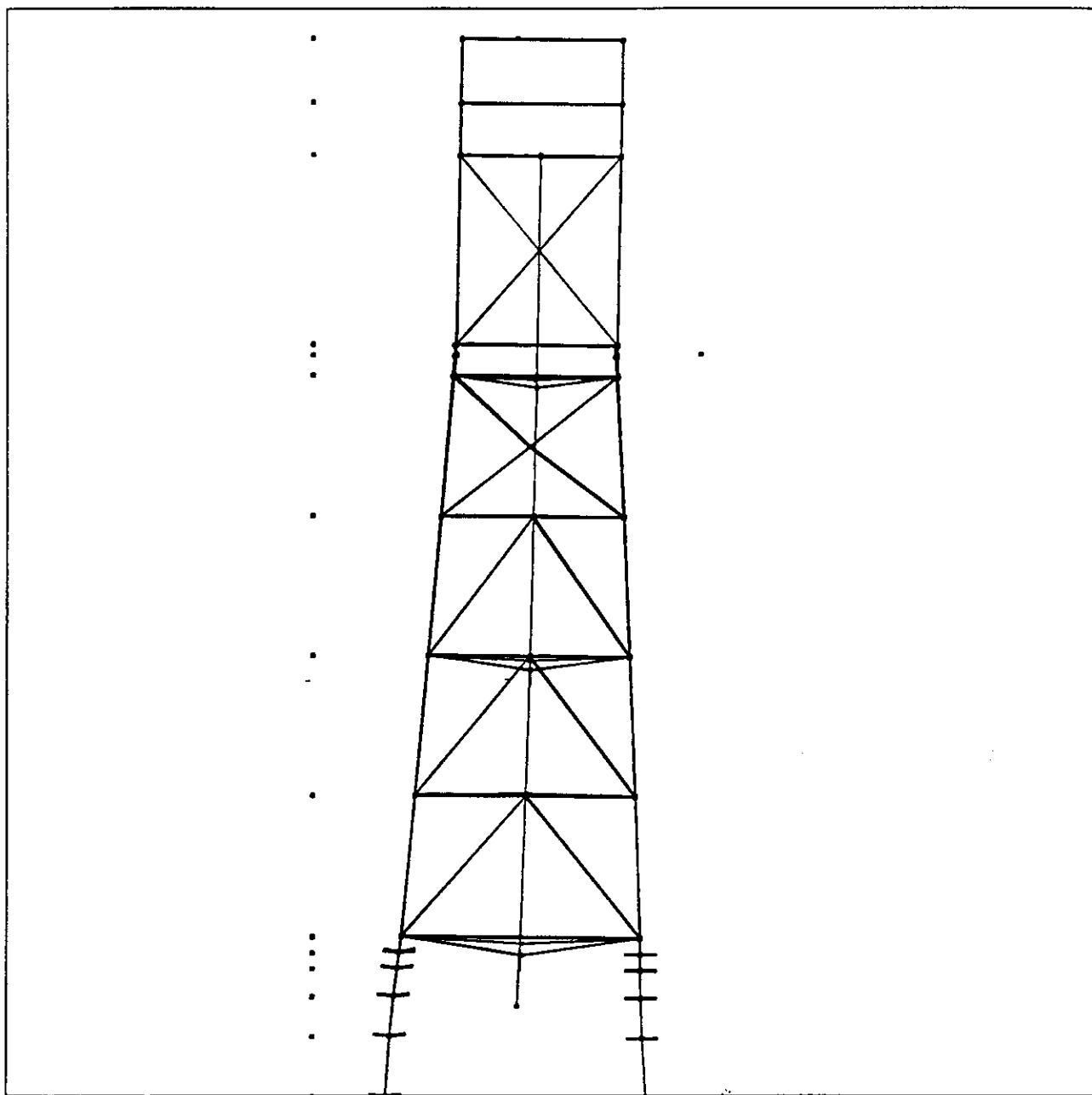
CAP - Pushover Load Ratio

Wed Jan 5 08:57:34 1994



Double Foundation (iter 0)

Figure 4-9 Increased Foundation Strength Results



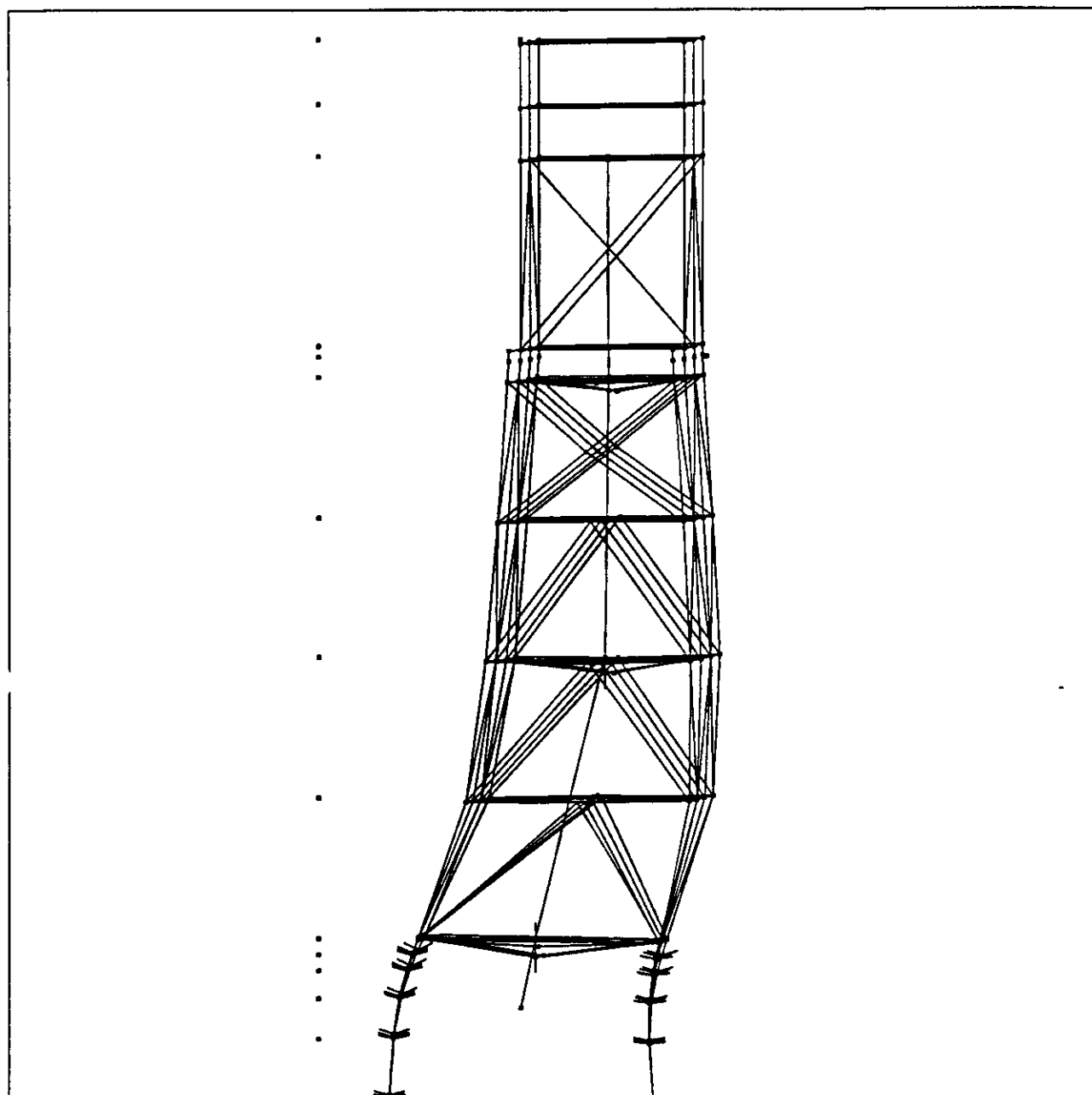
CAP  $\uparrow$  x

Double Foundation Model: Shape at Step 23

Inelastic Events Legend

—————	Elastic	.....	Strut Unloading
.....	Strut Residual	-----	Strut T/C Post Buckling
.....	Plastic Strut/NLTruss	.....	Beam Clmn Initial Yield
.....	Beam Clmn Fully Plastic		

**Figure 4-10 Increased Foundation: Load Ratio = 0.550**



Double Foundation Model: Shape at Step 45

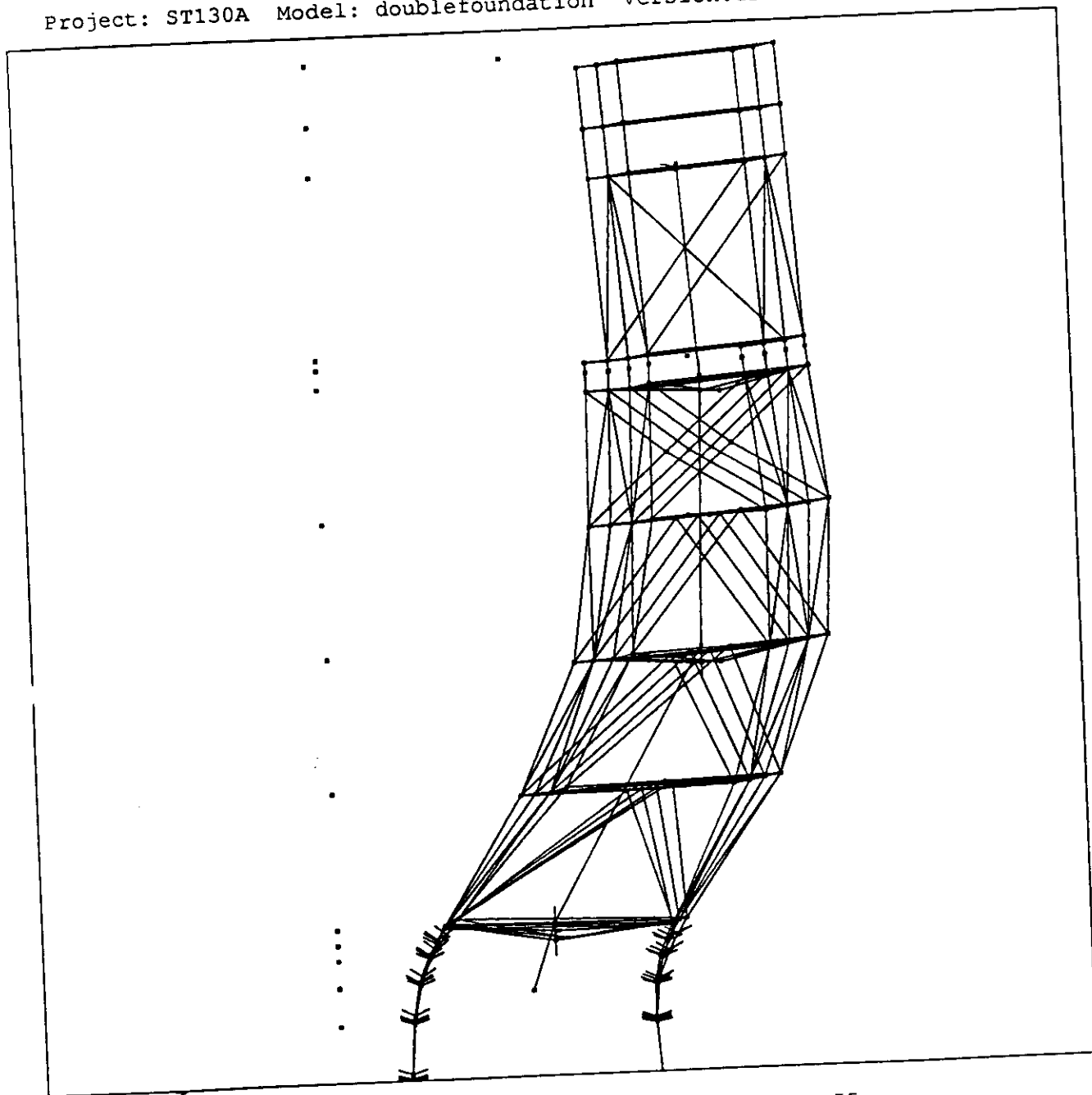
Inelastic Events Legend

—————	Elastic	.....	Strut Unloading
.....	Strut Residual	.....	Strut T/C Post Buckling
.....	Plastic Strut/NLTruss	.....	Beam Clmn Initial Yield
.....	Beam Clmn Fully Plastic		

Figure 4-12 Increased Foundation: Load Ratio = 1.100



Project: ST130A Model: doublefoundation Version: TMu Jan 20 16:59:55 1994 \*



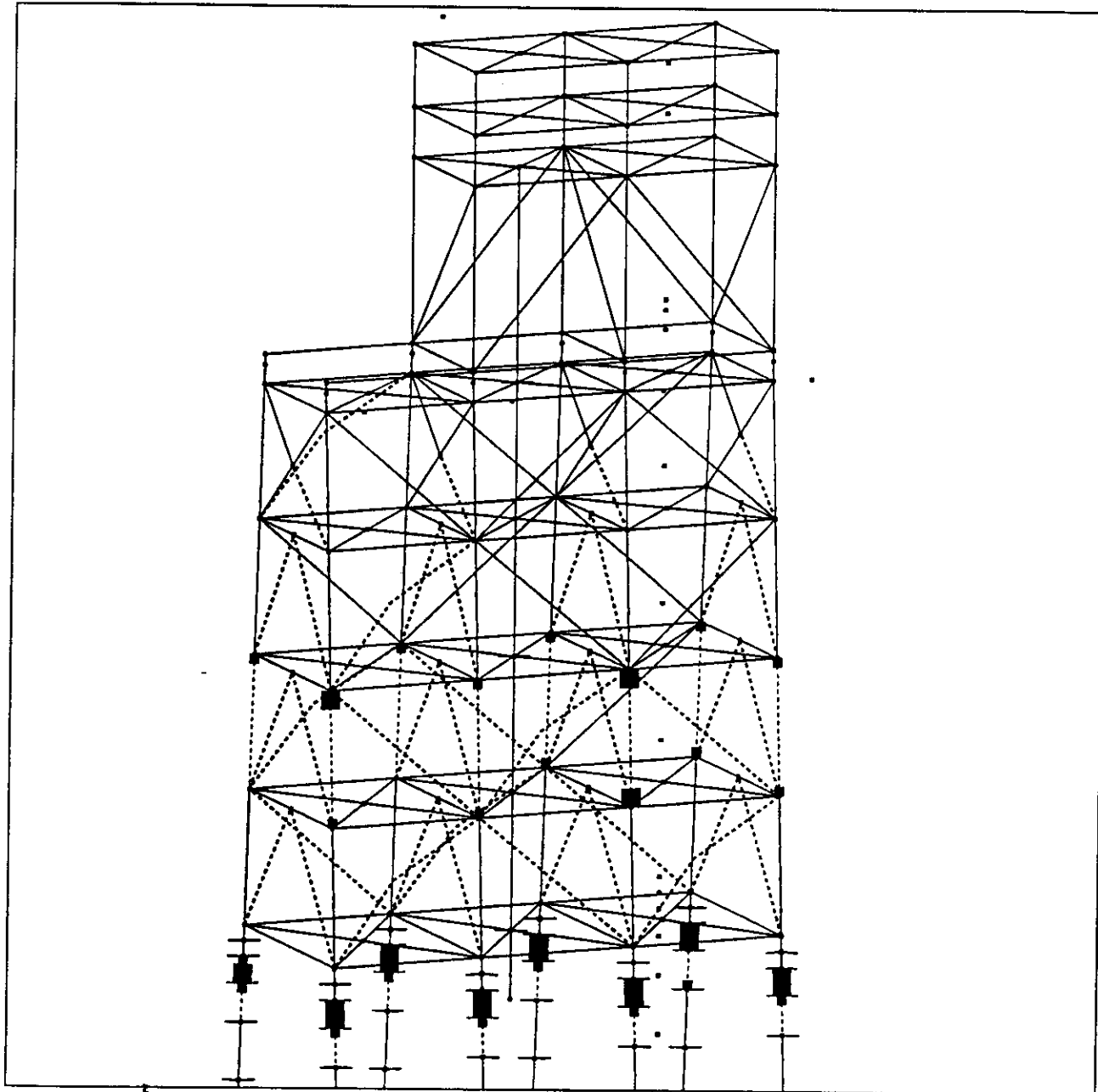
Double Foundation Model: Shape at Step 55

Inelastic Events Legend

— Elastic  
 ..... Strut Residual  
 ..... Plastic Strut/NLTruss  
 ..... Beam Clmn Fully Plastic

..... Strut Unloading  
 ..... Strut T/C Post Buckling  
 ..... Beam Clmn Initial Yield

Figure 4-13 Increased Foundation: Load Ratio = 1.350



Weak Leg Model: Failed Components

Inelastic Events Legend

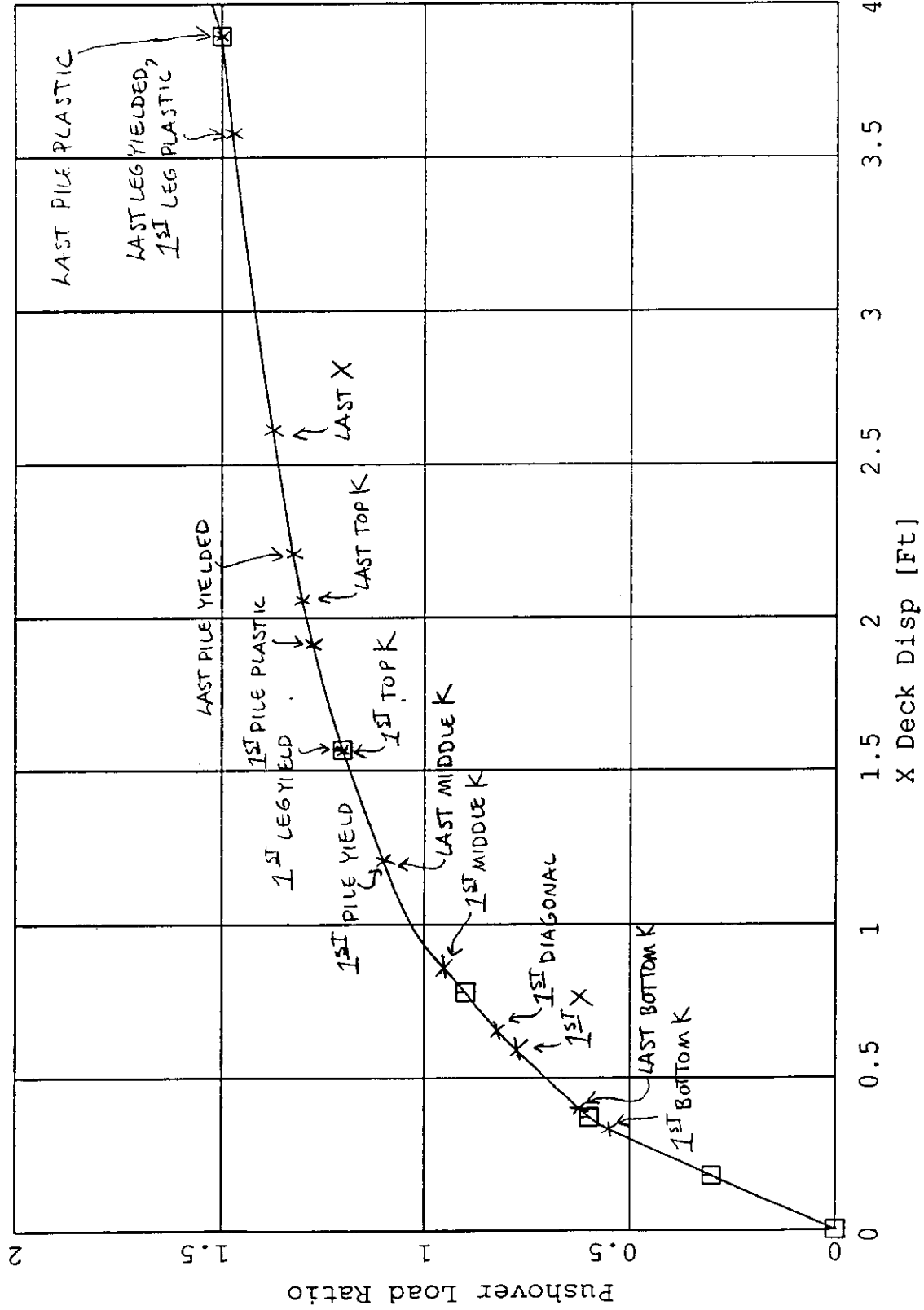
————	Elastic	.....	Strut Unloading
-----	Strut Residual	-----	Strut T/C Post Buckling
.....	Plastic Strut/NLTruss	.....	Beam Clmn Initial Yield
.....	Beam Clmn Fully Plastic		

**Figure 4-16 Fixed Base - Weak Leg**

Project: ST130A Model: weakleg Version: 1

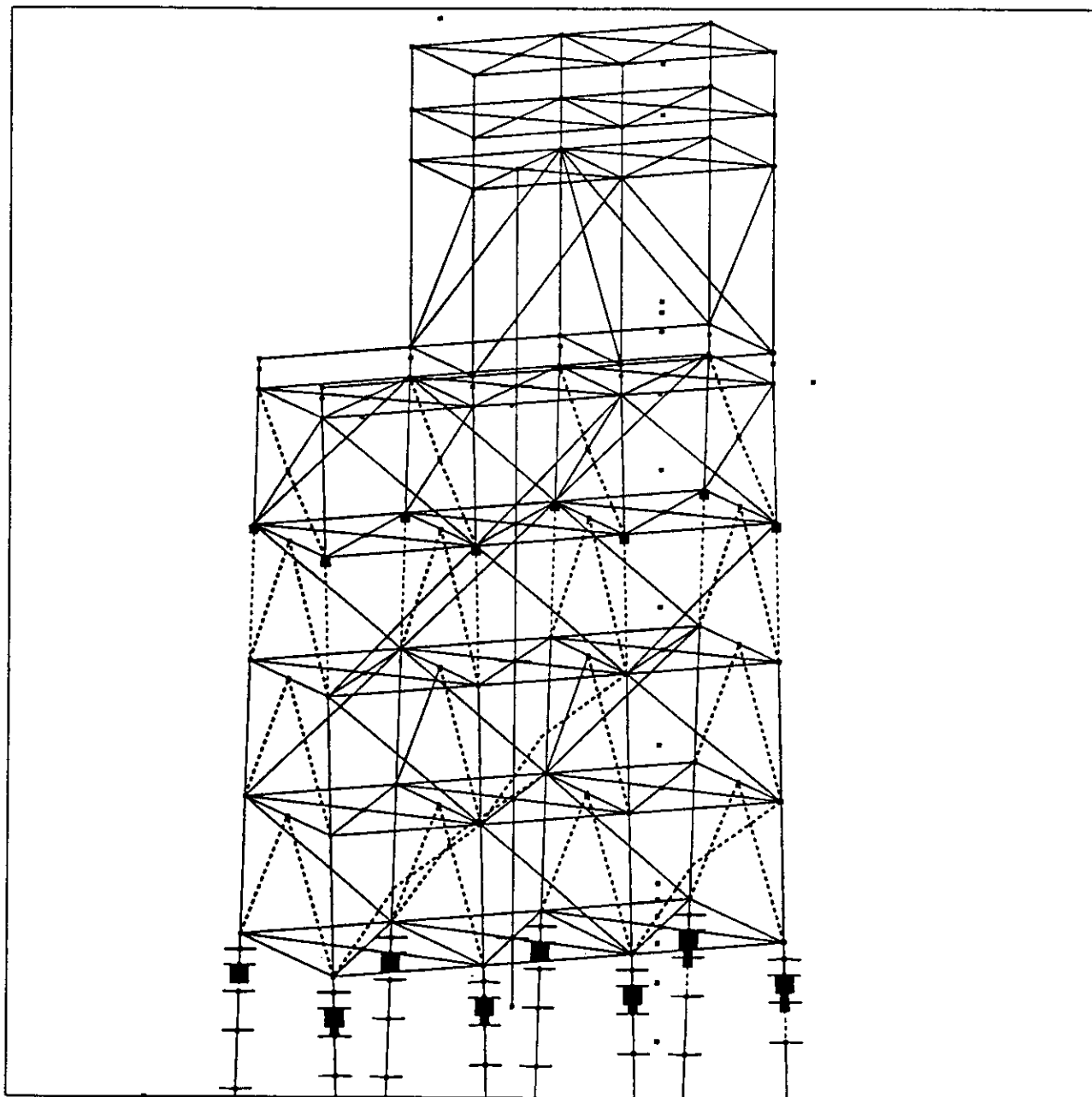
CAD - Pushover Load Ratio

Wed Jan 5 23:30:05 1994



Weak Legs (iter 0)

Figure 4-17 Weak Leg Model Results



CAP

### Brittle Brace Model: Failed Components

#### Inelastic Events Legend

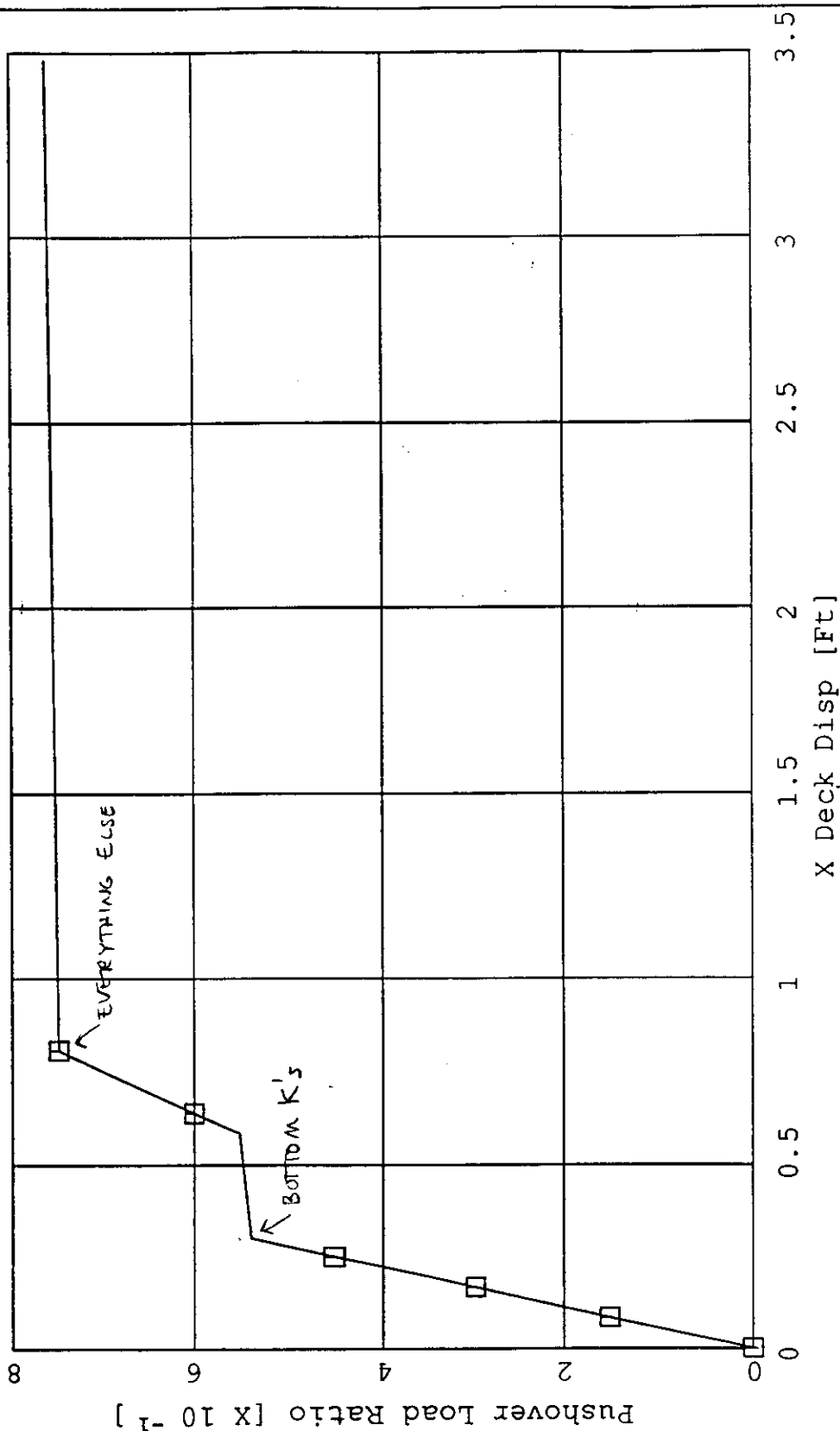
————	Elastic	.....	Strut Unloading
.....	Strut Residual	.....	Strut T/C Post Buckling
.....	Plastic Strut/NLTruss	.....	Beam Clmn Initial Yield
.....	Beam Clmn Fully Plastic		

**Figure 4-18 Brittle Brace Model Failed Components**

Project: ST130A Model: plaskbas Version: 1

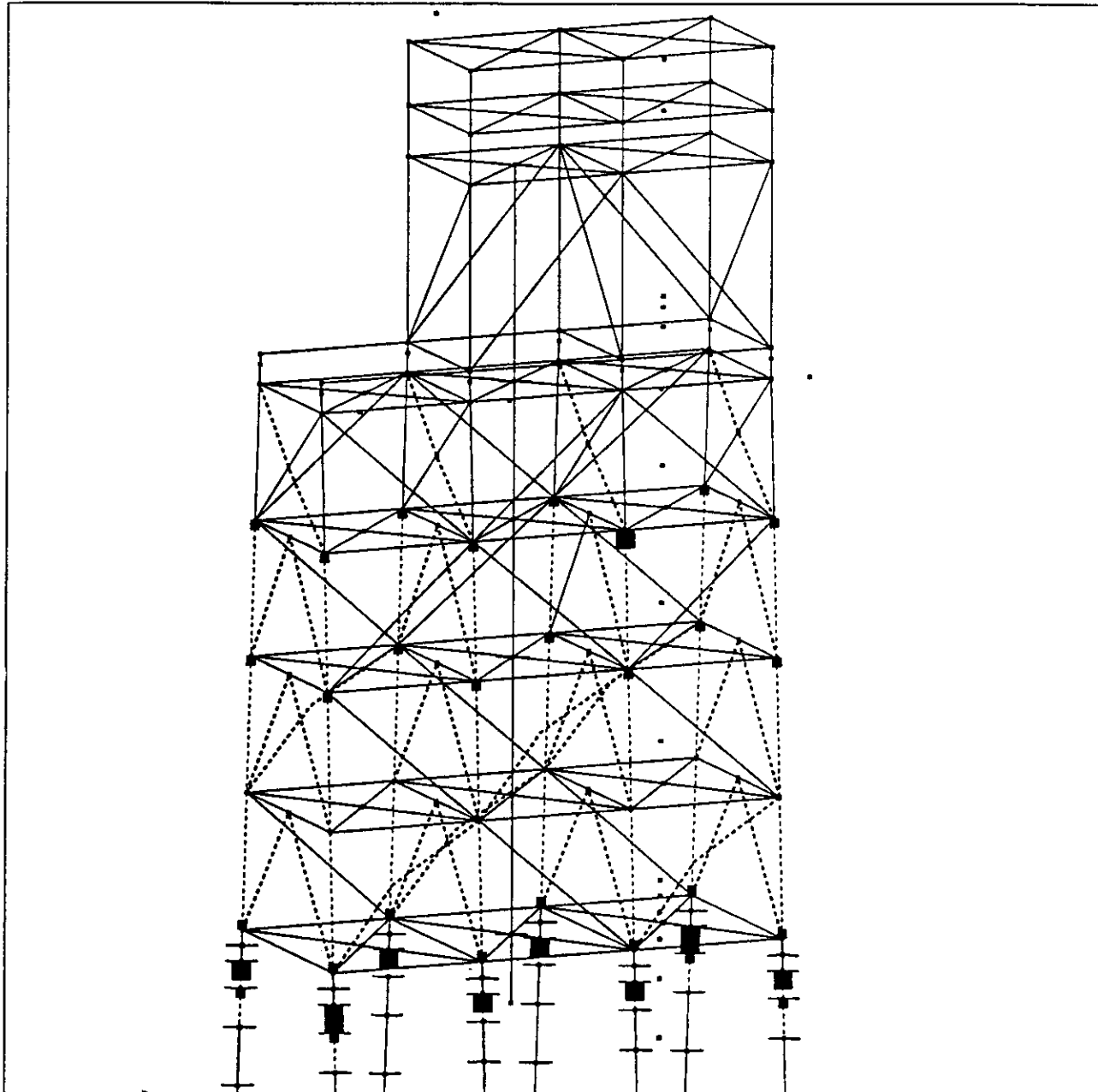
Sat Jan 8 09:34:09 1994

CAP - Pushover Load Ratio



Plastic Ks, full leg/pile

Figure 4-19 Brittle Brace Model Results



CAP

Brittle K, Weak Leg: Step 30 (includes previous)

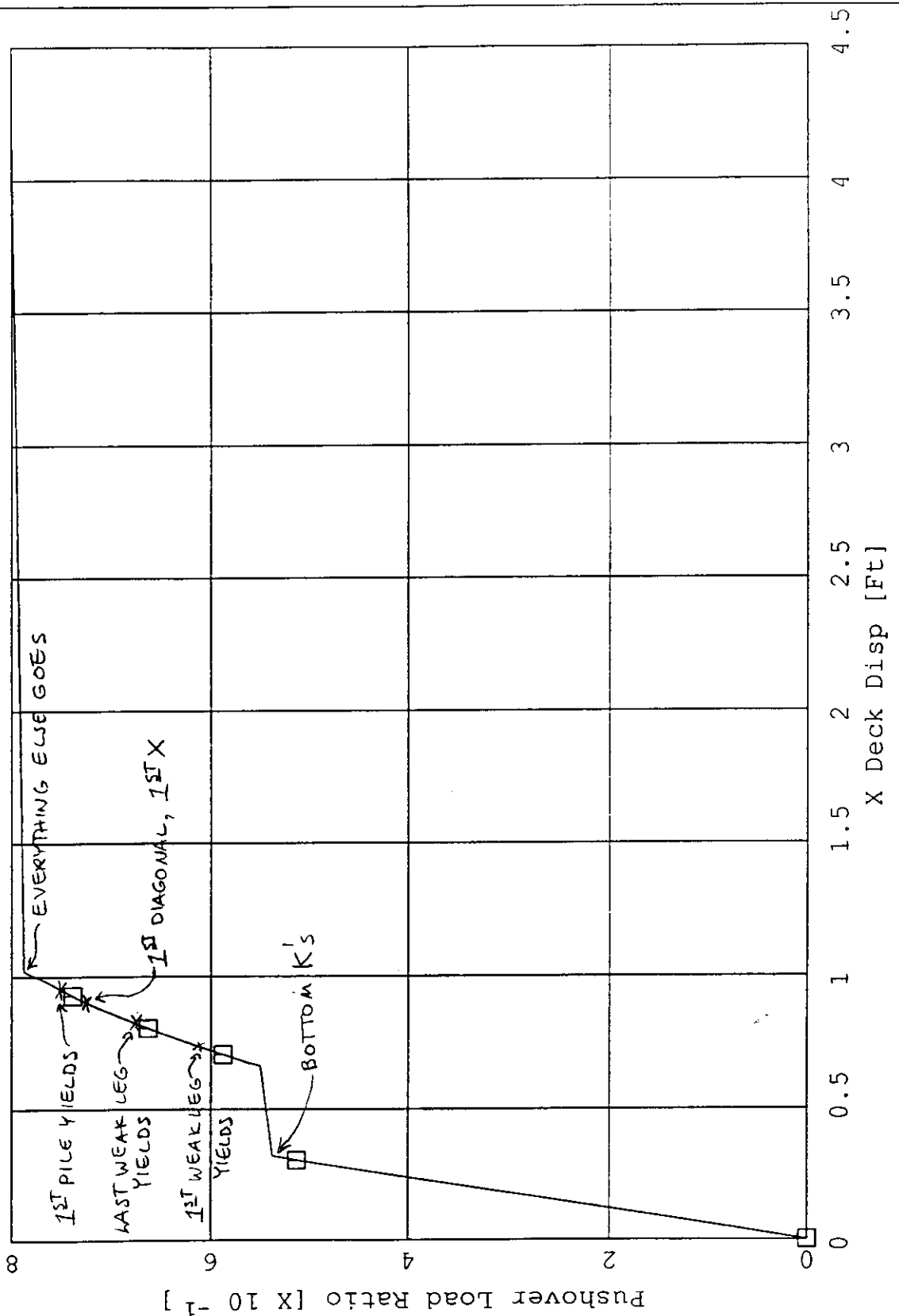
## Inelastic Events Legend

————	Elastic	.....	Strut Unloading
.....	Strut Residual	.....	Strut T/C Post Buckling
.....	Plastic Strut/NLTruss	.....	Beam Clmn Initial Yield
.....	Beam Clmn Fully Plastic		

**Figure 4-20 Brittle Brace Weak Leg Model Failed Components**

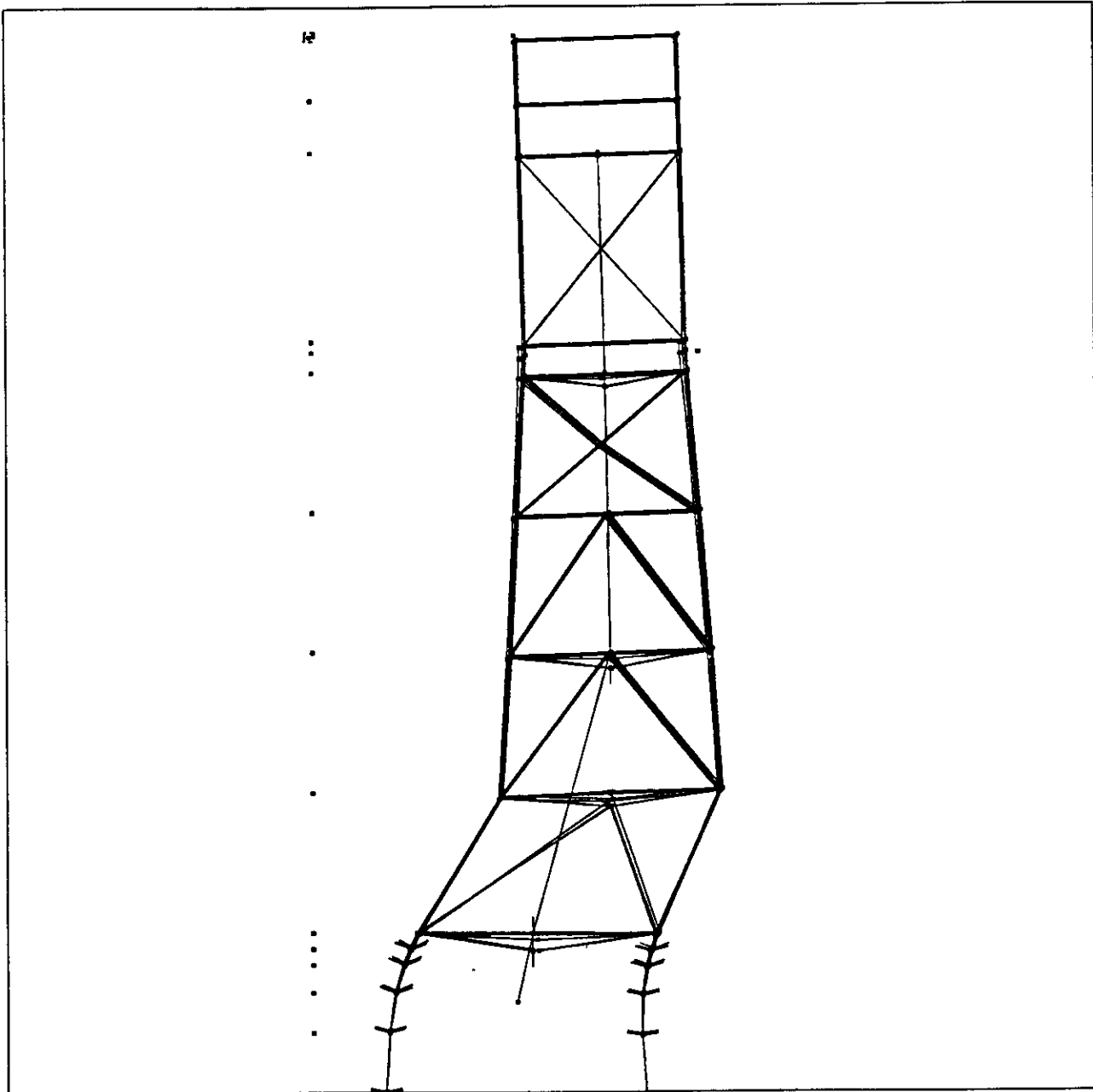
Fri Jan 21 08:39:29 1994

CAD - Pushover Load Ratio



Brittle K, Weak Leg Model

**Figure 4-21 Brittle Brace Weak Leg Results**

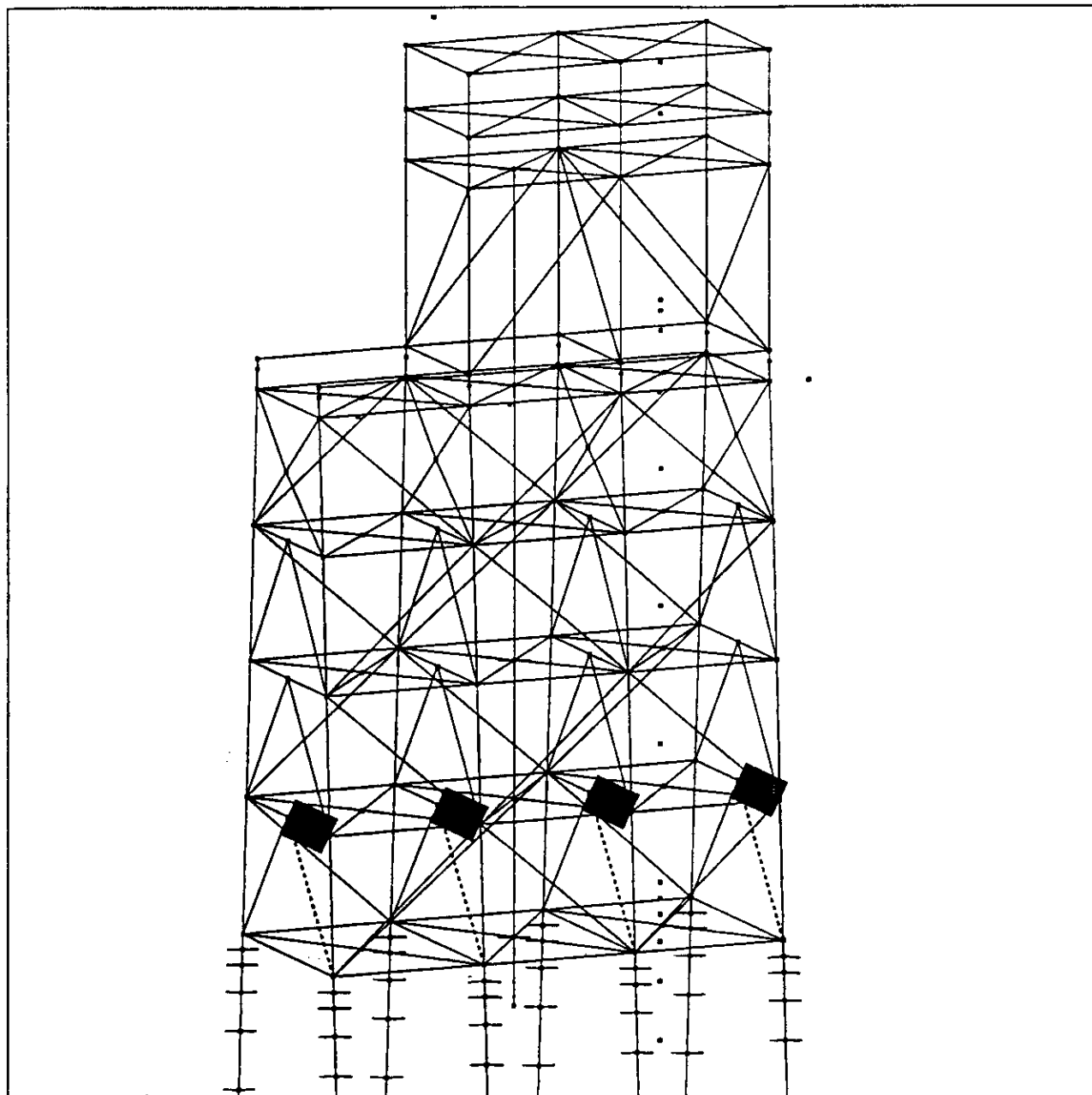


CAP  $\vec{L} \rightarrow x$

Brittle K, Weak Leg: Shape at Step 29

**Figure 4-22 Brittle Brace Weak Leg Shape Near Collapse**





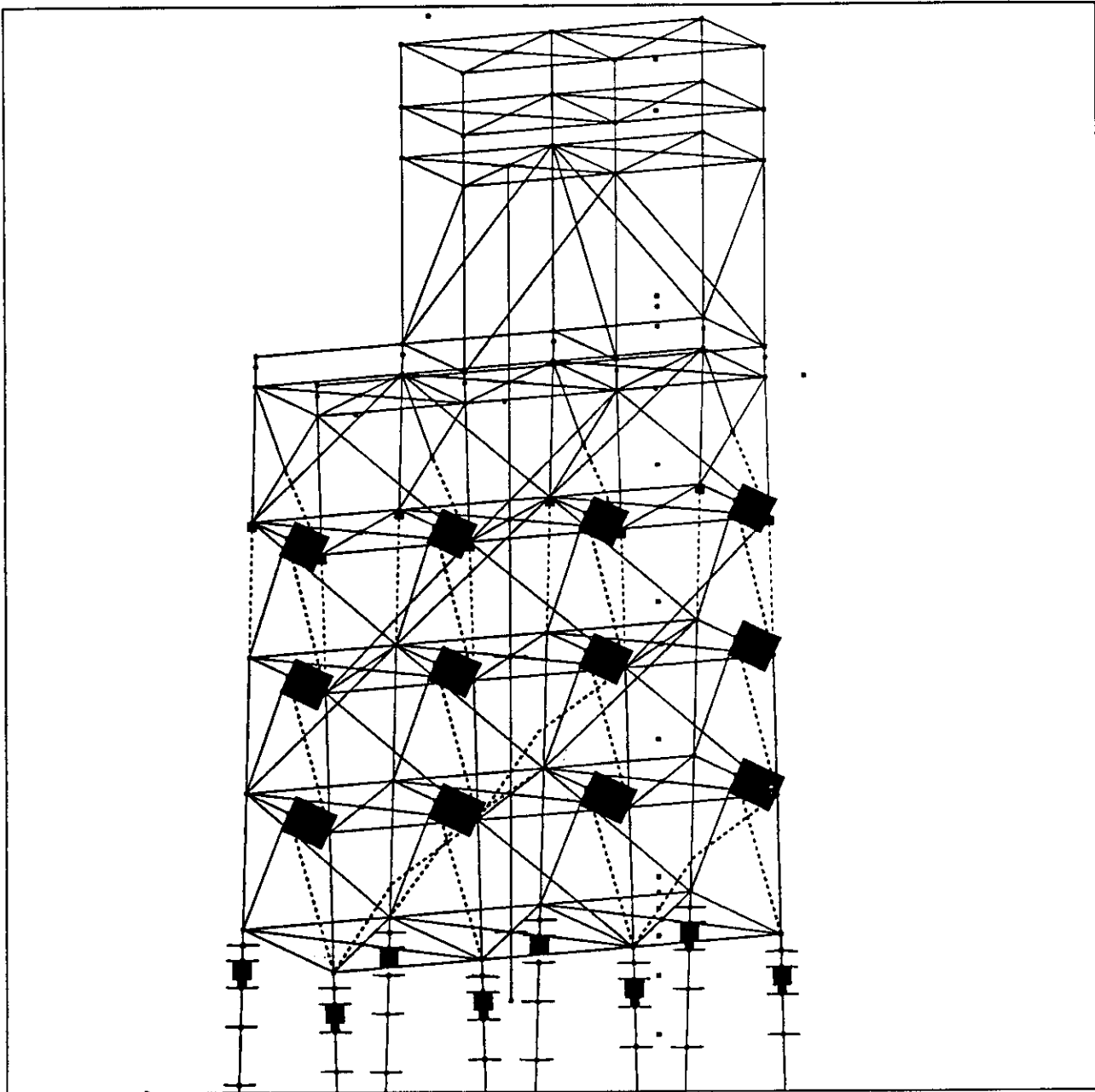
CAP

Brittle Joints (Full Leg/Pile): Step 10

Inelastic Events Legend

————	Elastic	.....	Strut Unloading
.....	Strut Residual	-----	Strut T/C Post Buckling
.....	Plastic Strut/NLTruss	.....	Beam Clmn Initial Yield
.....	Beam Clmn Fully Plastic		

**Figure 4-23 Brittle Joint Model Failed Components**



CAP

Brittle Joints (Full Leg/Pile): Step 23

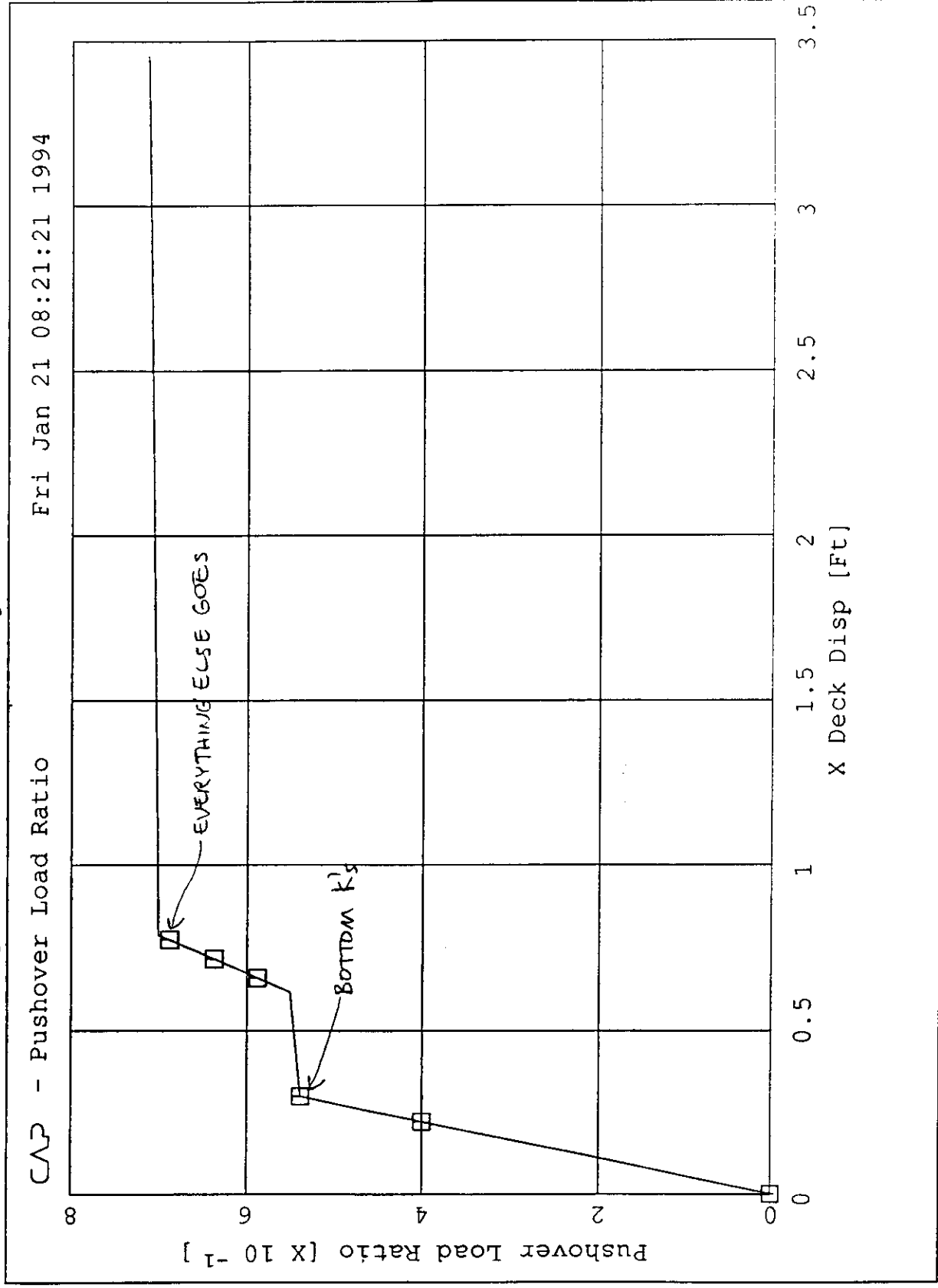
## Inelastic Events Legend

————	Elastic	.....	Strut Unloading
.....	Strut Residual	.....	Strut T/C Post Buckling
.....	Plastic Strut/NLTruss	.....	Beam Clmn Initial Yield
.....	Beam Clmn Fully Plastic		

**Figure 4-24 Brittle Joint Model Shape Near Collapse**

Project: ST130A Model: jntKbas Version: 2

Fri Jan 21 08:21:21 1994



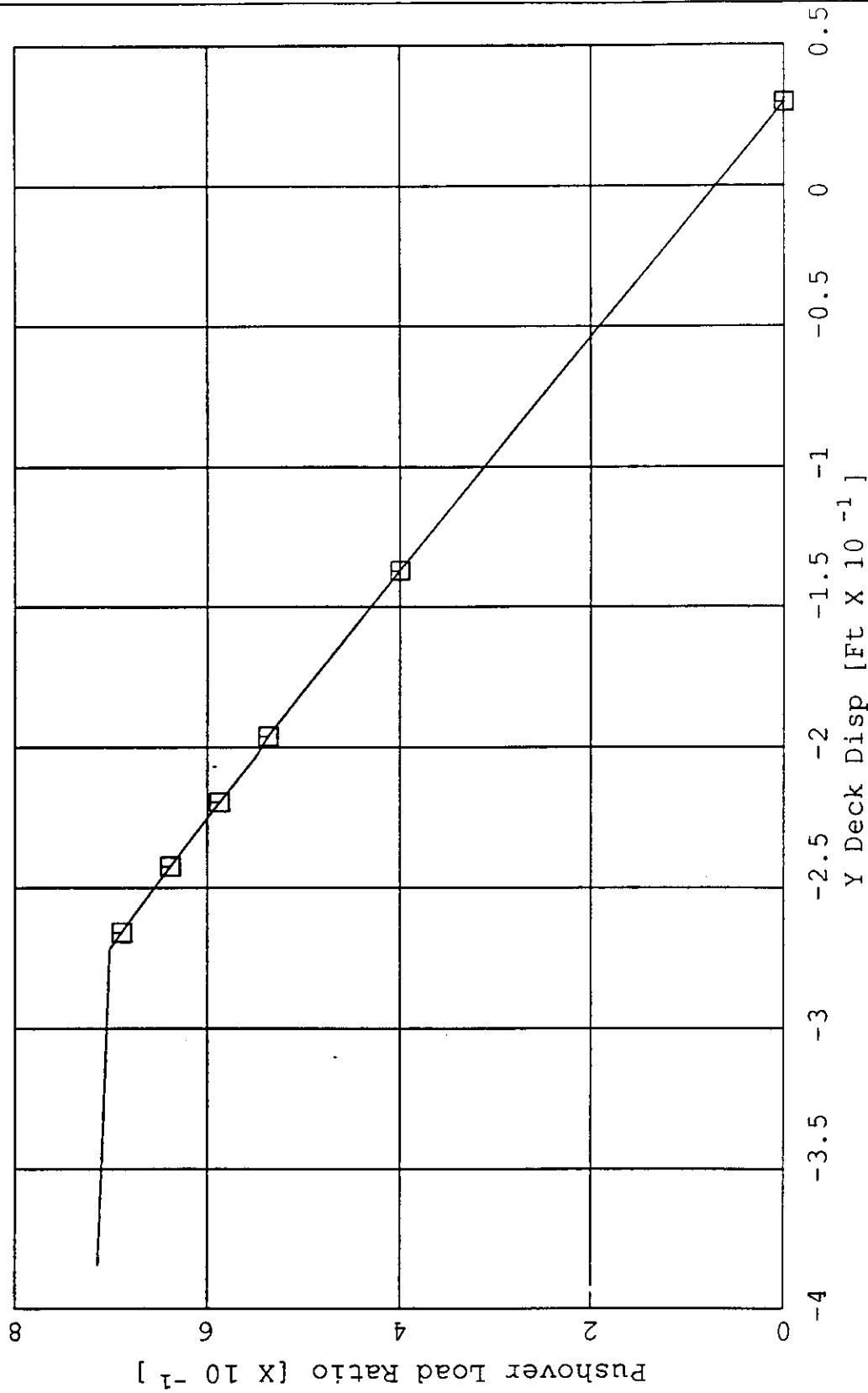
Brittle Joints (Full Leg/Pile)

Figure 4-25 Brittle Joint Model Results - X Direction

Project: ST130A Model: jntKbas Version: 2

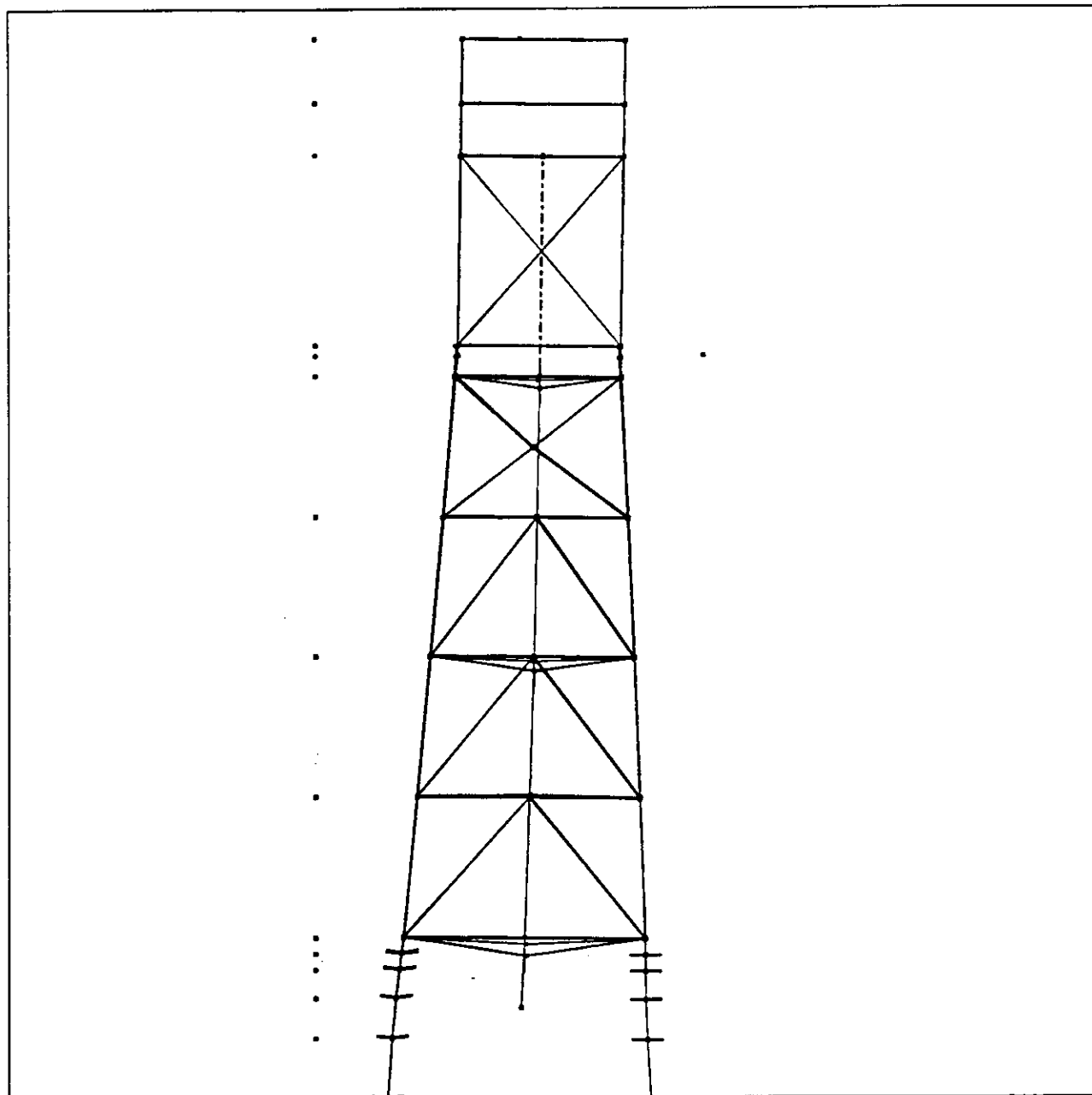
Fri Jan 21 08:21:21 1994

CAP - Pushover Load Ratio



Brittle Joints (Full Leg/Pile)

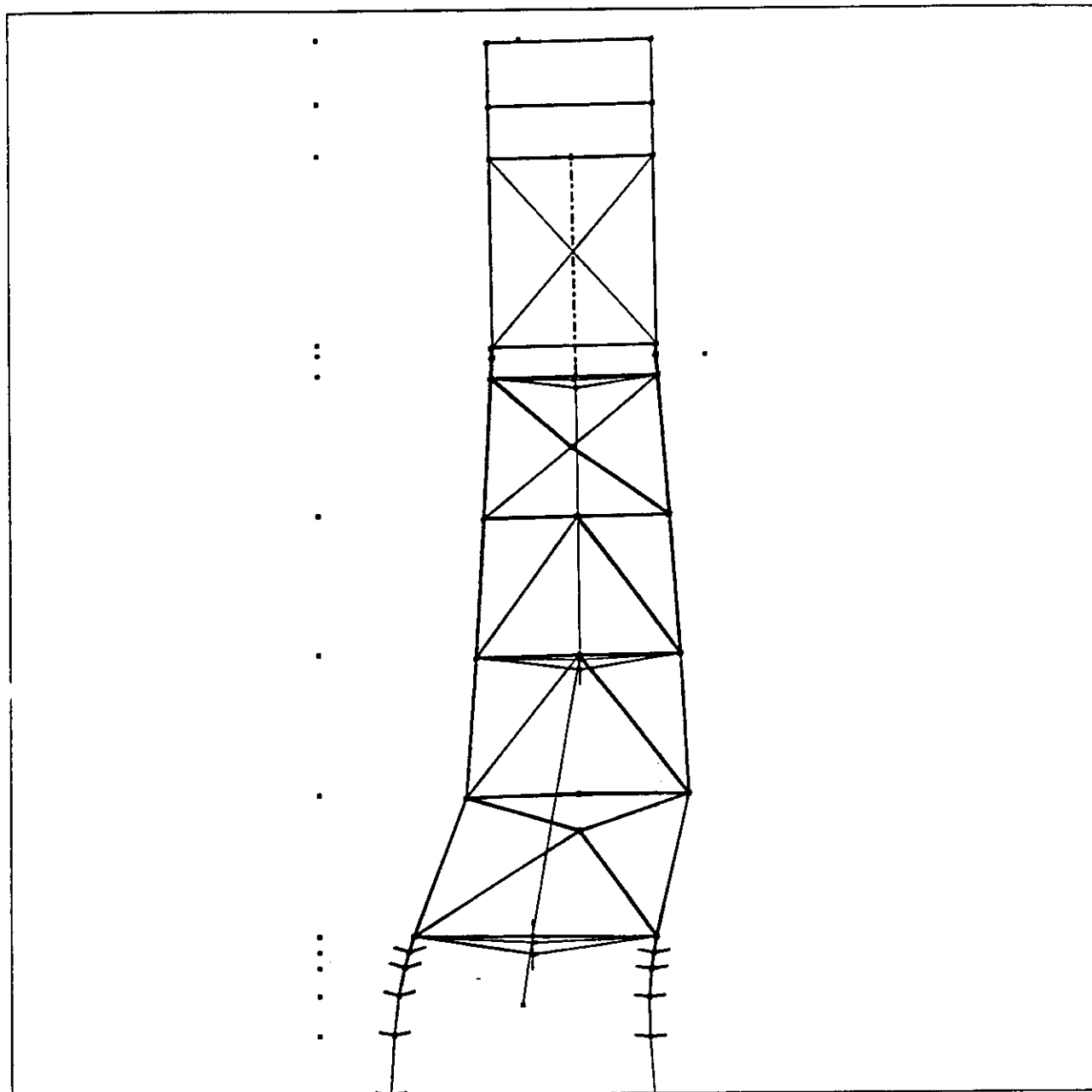
Figure 4-26 Brittle Joint Model Results - Y Direction



CAP  $\uparrow$  x

Brittle Joints: Shape at Step 9

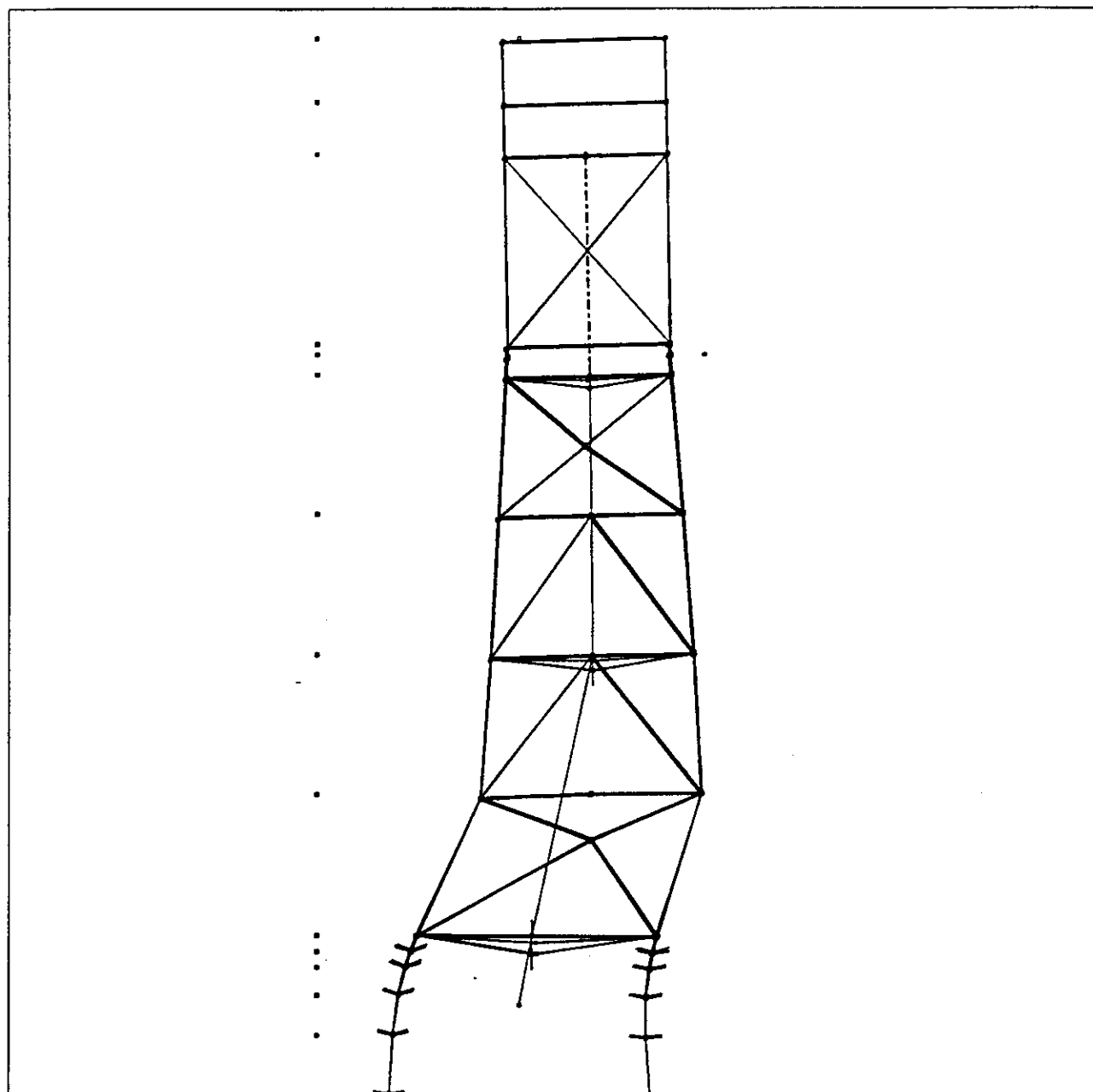
**Figure 4-27 Brittle Joint Model: Load Ratio = 0.5375**



CAP  $\uparrow$   $\rightarrow$  x

Brittle Joints: Shape at Step 10

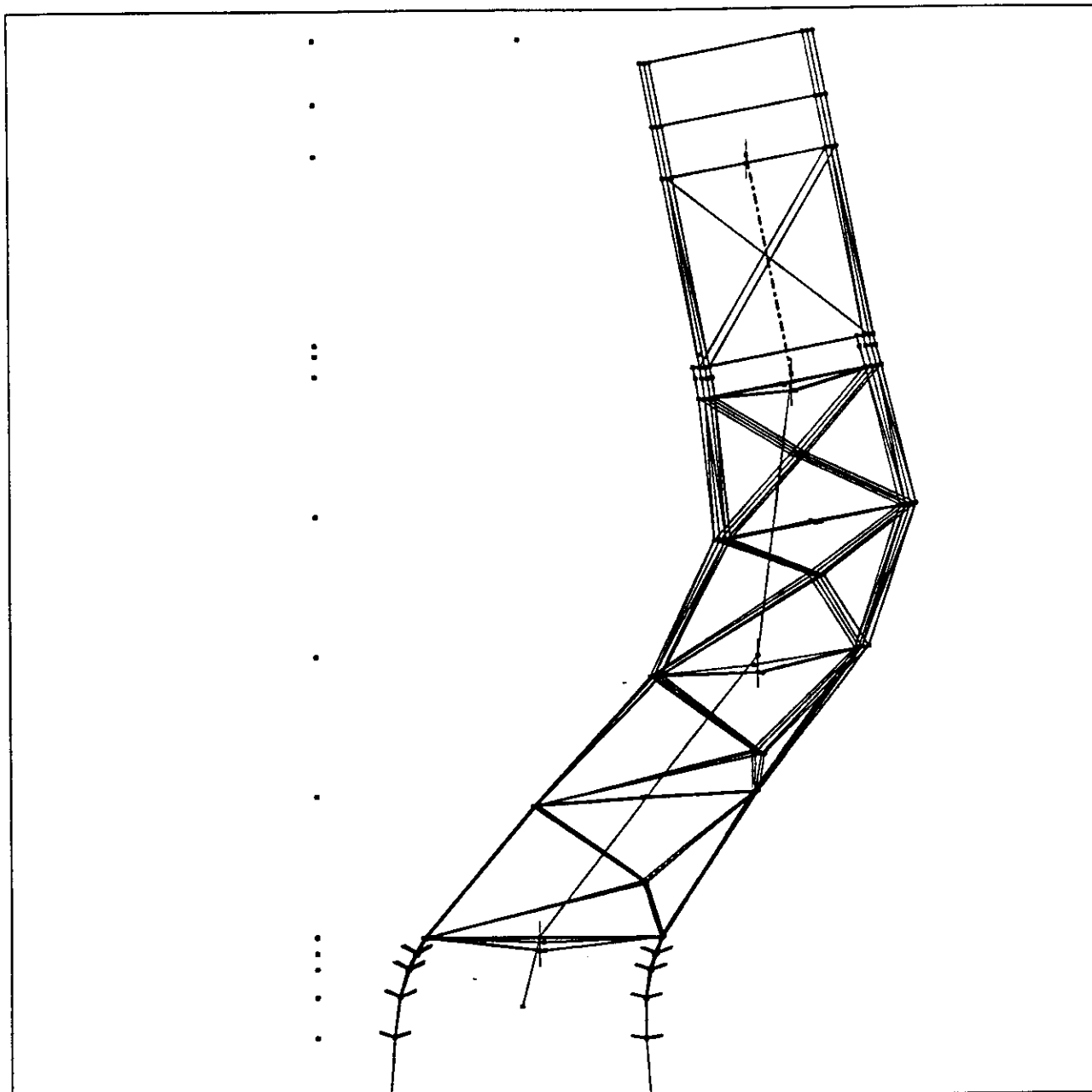
**Figure 4-28 Brittle Joint Model: Load Ratio = 0.5500**



CAP  $\uparrow$  x

Brittle Joints: Shape at Step 22

**Figure 4-29 Brittle Joint Model: Load Ratio = 0.7000**

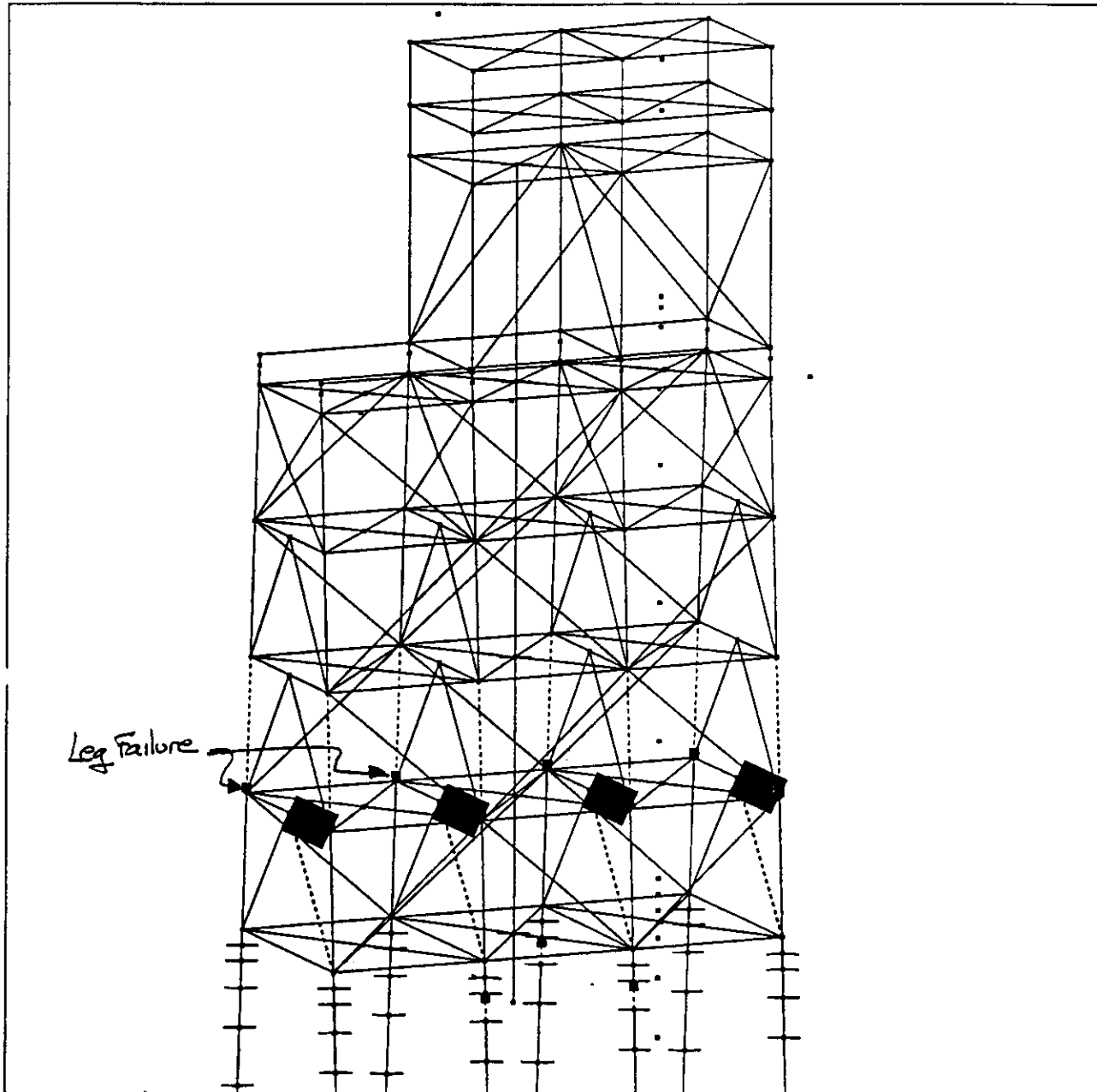


CAP  $\uparrow$   $\rightarrow$  x

Brittle Joints: Shape at Step 23

**Figure 4-30 Brittle Joint Model: Load Ratio = 0.7125**





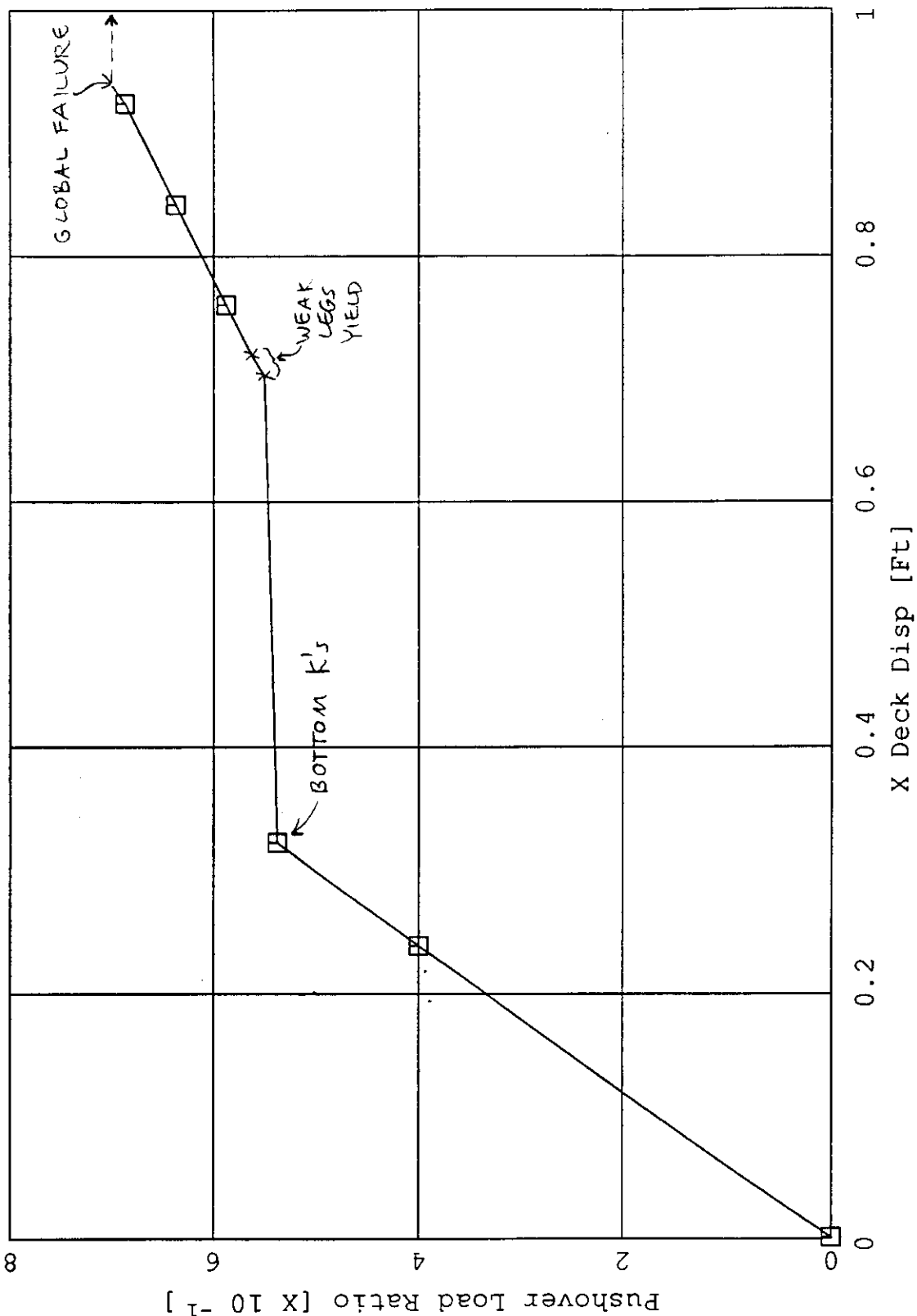
CAP

Brittle Joints, Weak Legs: Step 22

Inelastic Events Legend

————	Elastic	.....	Strut Unloading
.....	Strut Residual	-----	Strut T/C Post Buckling
.....	Plastic Strut/NLTruss	.....	Beam Clmn Initial Yield
.....	Beam Clmn Fully Plastic		

**Figure 4-31 Brittle Joint Weak Leg Model Component Failures**



Brittle Joints, Weak Legs

Figure 4-32 Brittle Joint Weak Leg Model Results

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## SECTION 5 ST 151 K ANALYSIS

### 5.1 INTRODUCTION

A CAP model was created of the ST 151 K structure, based on a SACS model provided by Chevron and updated by PMB for the Andrew JIP. The platform was also updated based upon field observations during the inspection. The platform was assumed to have 12 conductors, each 30 inches in diameter, located in the central bay. Pushover analyses were performed using environmental loads generated based on the most severe hour of Andrew. A set of analyses of increasing sophistication were performed and their results compared in order to determine the effect of analysis assumptions.

The following analyses were performed:

- Basic Model  
Soil strength per API RP 2A, elasto-plastic joint failure.
- Increased Foundation Strength Model  
Same as the basic model, except that the soil strength is doubled. See Section 4.1. These soil properties are used for all subsequent analyses.
- Brittle K-Joints  
Same as the Increased Foundation model except the chord and brace of the K-joints are modeled to fail in a brittle mode.

### 5.2 DESCRIPTION OF MODEL

A plot of the model is shown in Figure 5-1.

#### 5.2.1 Substructure

The substructure was modeled explicitly, with the following exceptions and comments:

- Node cans were neglected (except for computing joint strengths).
- Risers and caissons were neglected.

### 5.2.2 Superstructure

Primary members in the superstructure were modeled explicitly. The deck was modeled approximately with effective member sizes chosen to reflect the estimated value of deck stiffness and weight. The deck weight was 500 kips.

### 5.2.3 Conductors

The twelve conductors were modeled explicitly. Equivalent linear beams were substituted for conductors below the mudline.

### 5.2.4 Piles and Soil

The piles are 30 inches in diameter and have a penetration of 180 feet.

Soil properties for this analysis model were based on the soil properties reported in a 1986 report by Fugro Inter, Inc. for block ST 151.

### 5.2.5 Element Types

The element types used in the model were the same as that described for the ST 130 A platform in Section 4.2.5.

## 5.3 ENVIRONMENTAL LOADS

The environmental loads were based on Andrew data for the worst hour of the storm, as follows:

- Maximum wave height = 60.85 ft
- Associated peak spectral period = 14 sec
- Associated current velocity = 3.46 ft/sec
- Maximum wind speed = 158 ft/s
- Environmental direction = 286 deg off true north, or 301 deg from platform north

The following assumptions were made, based on API RP2A, 20th edition:

- Storm tide and surge = 2 ft

- Current blockage factor = 0.85 (for diagonal direction)
- Current profile is constant
- Wave spreading factor = 0.88
- Conductor shielding factor = 0.90
- Marine growth profile is 3 in. on the diameter from mean low water to mudline
- Drag coefficient = 1.05 (0.65 for smooth members), inertia coefficient = 1.2 (1.6 for smooth members)

The wave reaches approximately six and one half feet into the cellar deck. Using API Preliminary Deck Force Guidelines (API, 1994) the calculated wave-in-deck load is 930 kip and is divided evenly at the cellar deck level among the eight legs. Wind loads calculated using API RP2A recommendations were insignificant (2.6% of wave-in-deck loads) and were neglected.

The combined wave and current load generated by CAP based on the above assumptions is 3935 kips.

The resulting environmental load used in the pushover analysis is 4865 kips. In the pushover analysis results, the "load ratio" is the actual pushover load divided by this load.

### **5.4 MEMBER AND JOINT CAPACITY CALCULATION AND MODELING**

Member and joint capacities were computed using the same approach as described for ST 130 A. Details are provided in Section 4.4.

### **5.5 ANALYSIS RESULTS**

#### **5.5.1 General**

A summary of capacities for the various analyses, in the form of pushover load ratio at failure, is presented in Table 5-1. A summary plot of the base shear as a function of deck lateral displacement for all analyses is shown in Figure 5-2. Analysis results presented in the following sections also include inelastic event plots and plots of the pushover load ratio versus lateral SRSS top deck displacement. Displaced shape plots will also be presented to show the displacement behavior. For all of the displaced shape plots, the displacements have been exaggerated 20 times

and the conductors have been removed for clarity.

### **5.5.2 Basic Model**

The capacity of the basic model is 0.66 times the Andrew environmental load. The displaced shape just prior to yield is shown in Figure 5-3. Yielded and failed elements with the corresponding displaced shapes for the last step are shown in Figure 5-4. The analysis terminated due to a large unbalance force in the piles, but it is clear that global failure would be caused by pile failure.

The pushover load plot is shown in Figure 5-5. The general behavior is a gradually, fairly smooth, decreasing stiffness, with no abrupt changes in behavior until the K-braces and piles fail. The first elements to fail are the compression K-braces in the interior bay in the first (bottom) level. This occurs at a load level of 0.66 times the total environmental load. The piles also begin to yield at a load ratio of 0.66. Although some K-braces fail, the failure mechanism for the platform is due to lateral failure of the piles.

### **5.5.3 Soils Improved 100%**

The capacity of the model with soils improved 100% is 1.00 times the Andrew environmental load or about 53% greater than the basic model's capacity. The displaced shape at first yield (bottom K braces) is shown in Figure 5-6. The displaced shapes for an intermediate step and the last load step are shown in Figures 5-7 and 5-8, respectively. The platform failure mechanism is still in the piles. All of the compression K-bracing in the top and bottom levels fail, as do the interior compression K-braces in the middle level.

The pushover load plot is shown in Figure 5-9, with major events indicated. The general behavior is that of a gradually, smoothly decreasing stiffness, with no abrupt changes in behavior until the piles become completely plastic. All of the compression K-braces in the bottom and top levels fail, as do the two interior K-braces in the middle level. The K-braces start to fail at 0.70 times the Andrew load and the piles start to fail at 0.80 times the total environmental load. Note that the increased soil strength resulted in a larger number of brace failures in the jacket prior to global failure of the piles. This is consistent with the similar ST 177 B structure where a majority of the jacket joints had failed, yet the platform was still standing after Andrew.

### **5.5.4 Soil Improved 100%, Brittle K-Joints**

The capacity of the model with brittle K-joints and soils improved 100% is 1.01 times the Hurricane Andrew environmental pushover load or about 53% greater than the basic model's capacity. The displaced shape and first yield are shown in Figure 5-10. Yielded and failed

elements and their corresponding displaced shapes for an intermediate load step and the last load step are shown in Figures 5-11, 5-12. The analysis terminated due to large displacements, but it appears that the failure mechanism has moved into the jacket legs. All of the compression K-braces fail, as have the horizontals that model the K-joints. There is also some initial yielding of the legs due to the large number of brittle K brace failures, meaning the legs must carry all of the lateral load.

The pushover load plot is shown in Figure 5-13, with major events indicated. The general behavior is that of a system that suddenly loses a large portion of stiffness when the bottom level K-joints fail at a load ratio of 0.63. The system loses considerable stiffness again when the middle and top level K-joints fail at a load ratio of 0.73.

This model is believed to be the best representation of the pre-Andrew platform based upon the previous ST 130 A study. The model predicts that the platform should have failed or at least suffered serious damage during Andrew. This does not compare to the actual post-Andrew platform, which was undamaged.

### **5.6 Conclusions**

As described in Section 2, the structure did not experience any observable failures due to Hurricane Andrew. The analyses performed on the various models of the structure all indicate that the ultimate capacity of the structure is less than or equal to the Hurricane Andrew environmental loading.

The earlier, less sophisticated analyses in the sequence would have predicted a global platform capacity driven by pile failure at an environmental load from 0.66 times the Andrew load (no increase in soil shear strength) to 1.00 times the Andrew load (soil shear strength increased 100%). This is not consistent with the platform's observed behavior, where the platform was undamaged.

The later analyses including revisions for increased soil strength and improved modeling of the joints. The 100% increase in the reported soil shear strength did improve the platform capacity from a pushover load ratio of 0.66 to 1.00, but even these results still lead to the conclusion that the platform should have suffered some significant damage, if not collapsed in Andrew.

Although the brittle joint modeling did not significantly affect the platform capacity (load ratio = 1.01); however, it did change the platform failure mechanism. Platform failure without the brittle K-braces was due to hinging in the piles. When the brittle failure K-joints are modeled, the failure mechanism moves into the jacket and even though the piles begin to yield they have not reached their ultimate strength. The brittle failure of the internal bracing forces the legs to



## MMS Platform Inspections and Analysis

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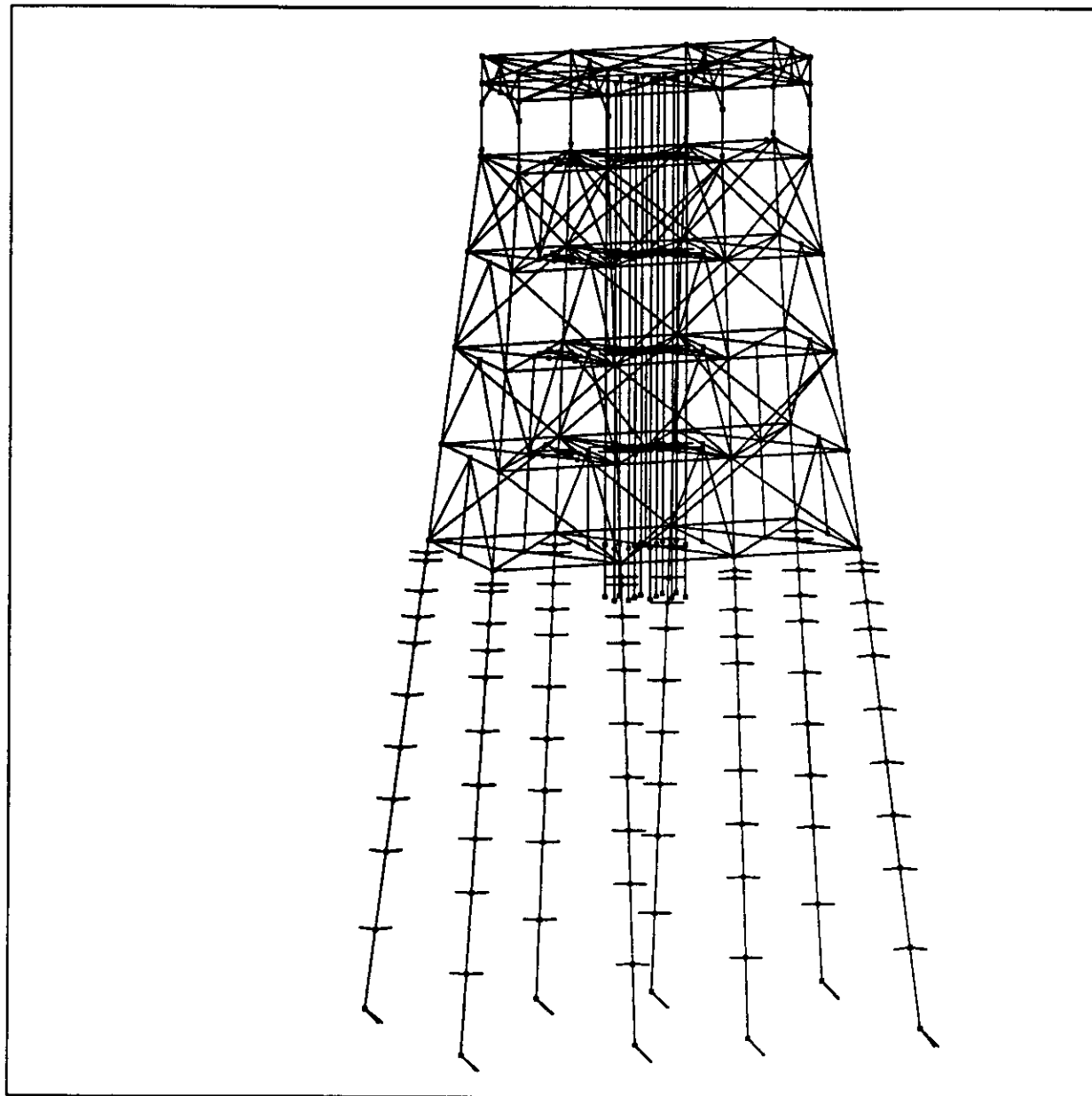
take a larger share of the load in bending instead of transferring the load down the jacket to the mudline and into the piles. Again, the expected damage predicted by this computer model, deemed the best approach based upon the ST 130 A results, does not reflect the observed platform behavior which was no damage.

It is generally unclear why this platform was undamaged. The best estimate computer models predict damage yet none occurred. One possible explanation is the wave in the deck loading. The computer model uses a hindcast wave height that implies dramatic loading in the deck as the maximum Andrew wave impacts the platform. The wave is assumed to be long crested resulting in wave forces acting across the entire platform. However, discussions with Chevron staff who inspected the platform immediately following Andrew indicate that although there were signs of waves acting on the deck, it did not appear that it was a long crested wave acting on the entire platform. Instead the wave damage to the deck was in certain areas only. Thus it is possible that the deck wave loads used in this analysis and which contribute significantly to the prediction of platform failure, are overestimated. This is an area of possible further study.

**Comparison of ST151K Pushover Analyses**

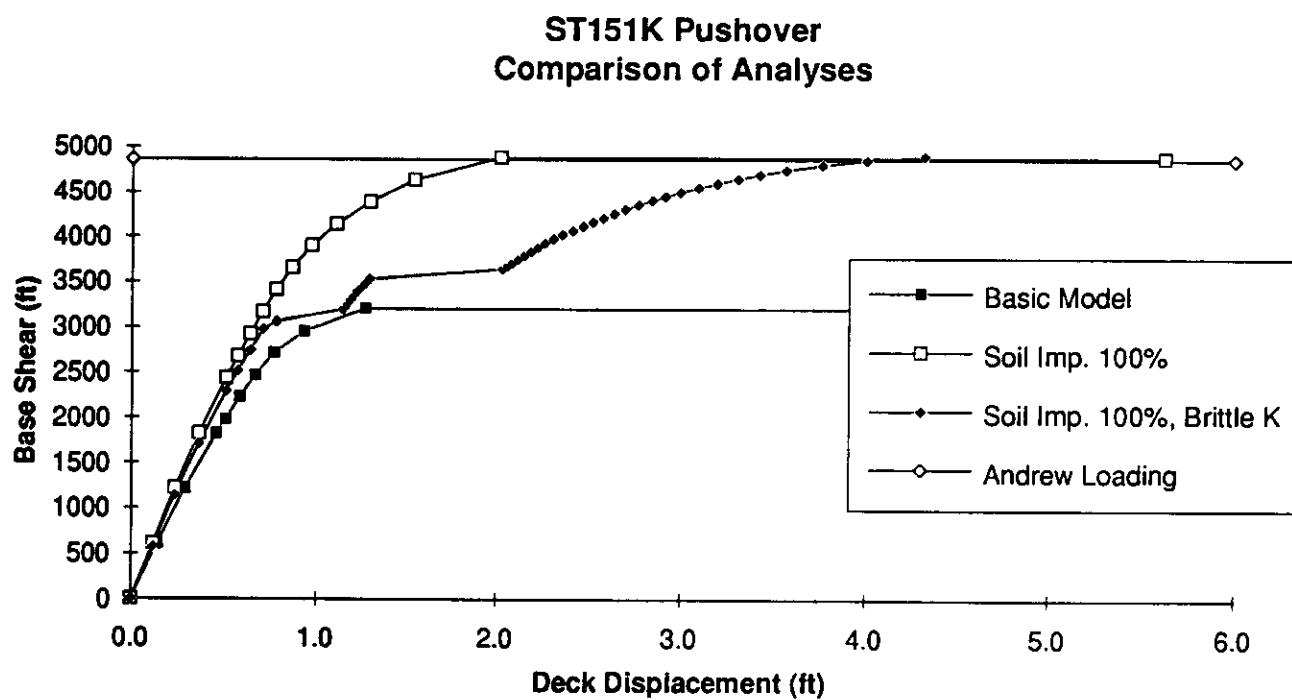
Run	Load Ratio
Basic Model	0.66
Soil Improved 100%	1.00
Soil Improved 100%, Brittle K-Joints	1.01

**Table 5-1 Summary of ST 151 K Results**

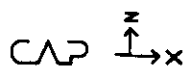
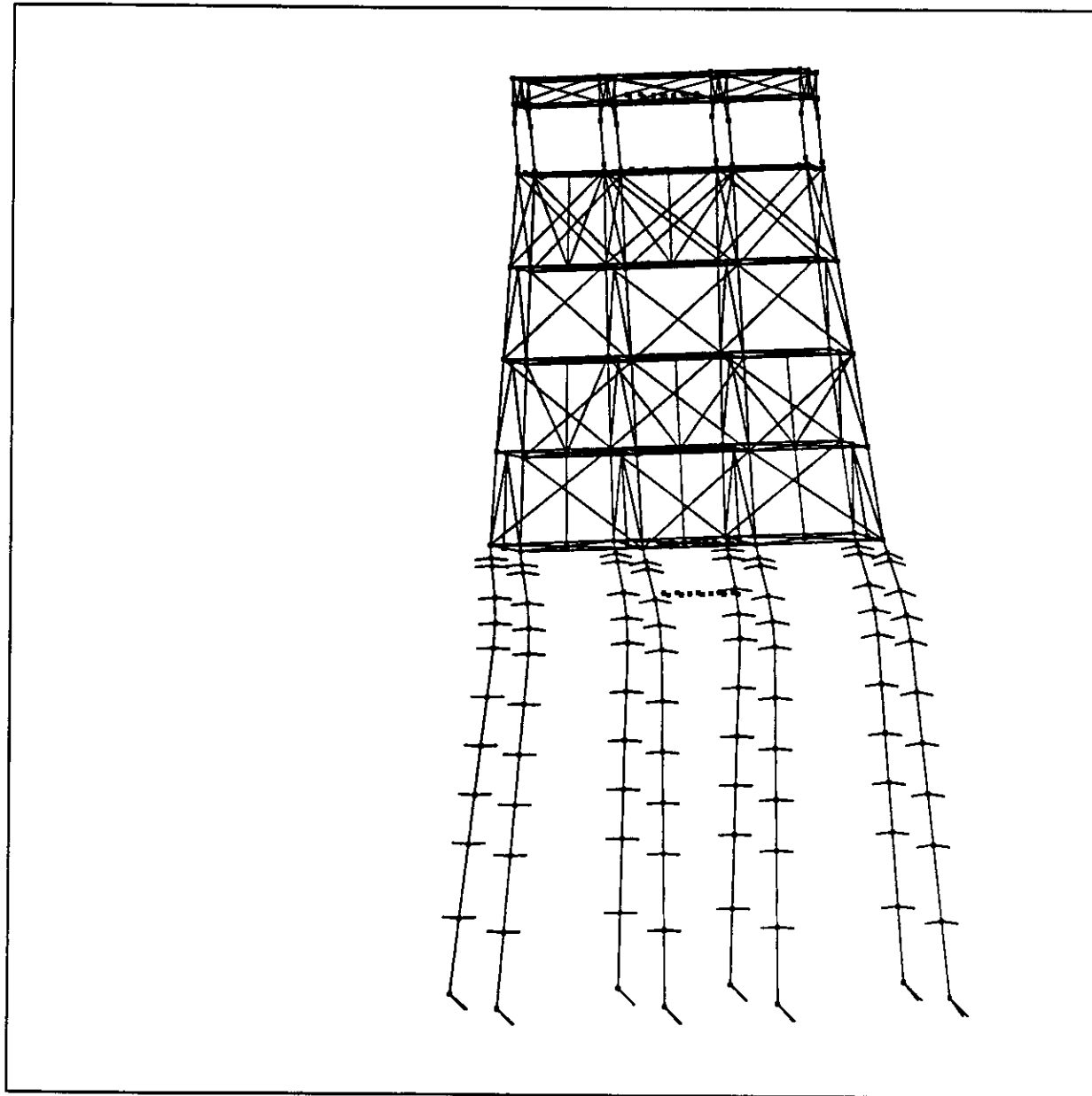


CAP  $\begin{matrix} \uparrow z \\ \rightarrow x \end{matrix}$

**Figure 5-1 ST 151 K CAP Computer Model**



**Figure 5-2 ST 151 K Pushover Comparison of Analysis Results**



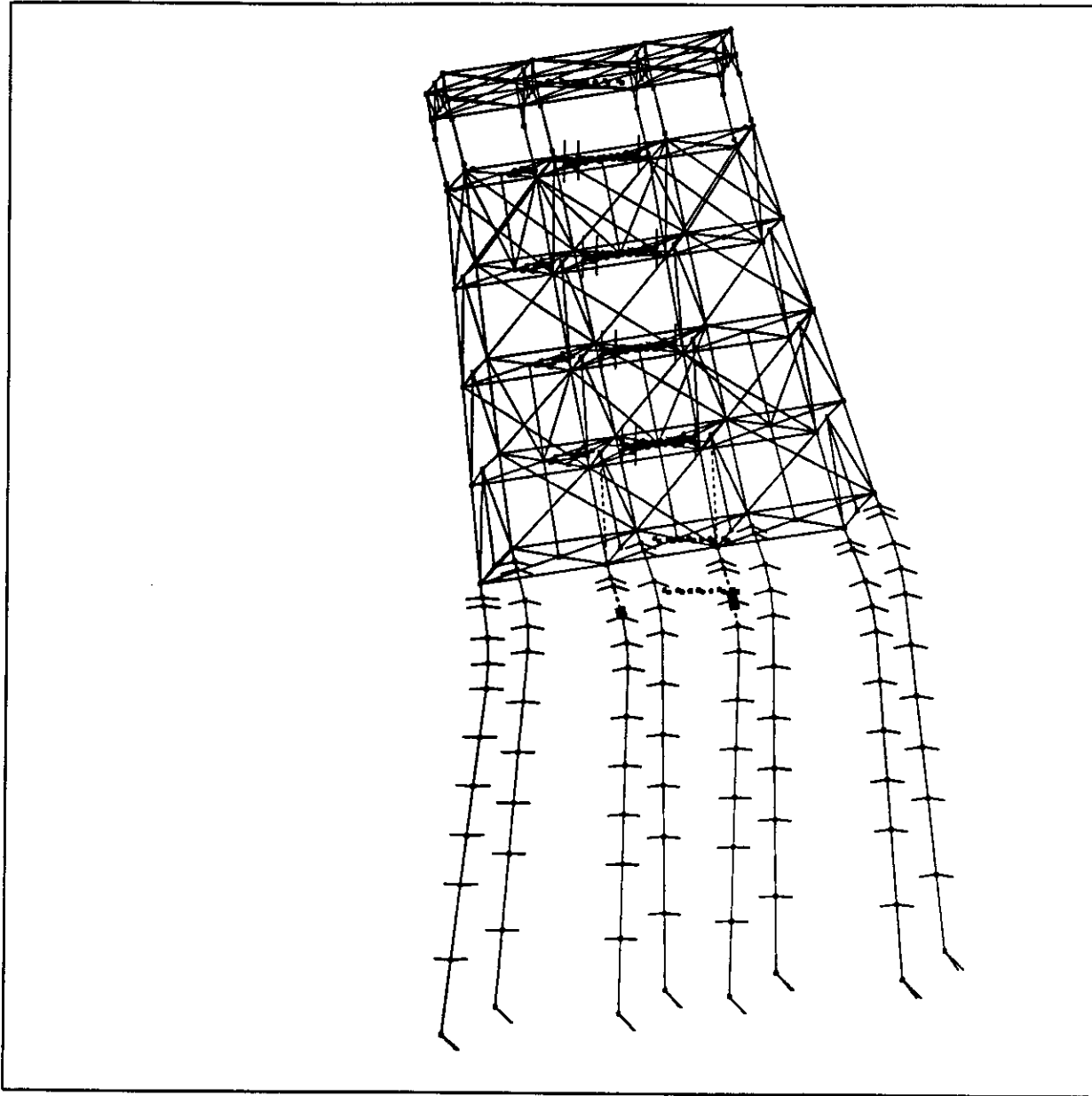
Basic Model

Inelastic Events Legend

— Elastic  
 - - - - - Strut Residual  
 . . . . . Plastic Strut/NLTruss  
 ——— Beam Clmn Fully Plastic

- - - - - Strut Buckling  
 ——— Strut Reloading  
 . . . . . Beam Clmn Initial Yield

**Figure 5-3 Basic Model: Load Step at First Yield**



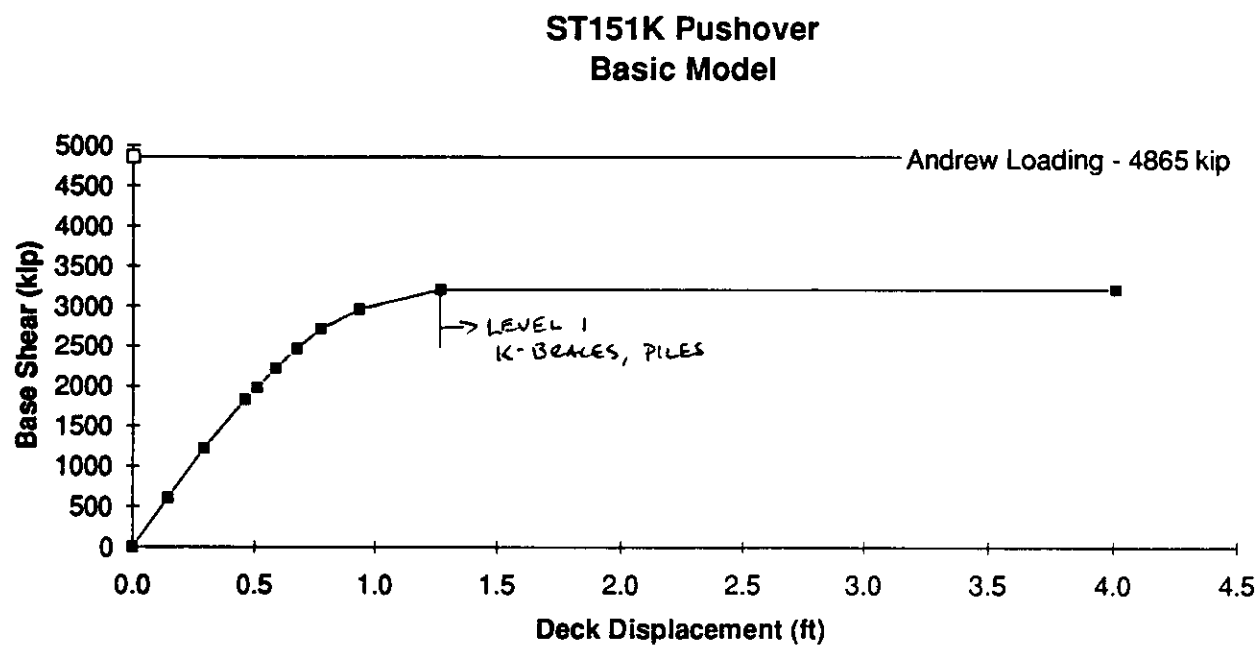
CAP  $\begin{matrix} \uparrow z \\ \rightarrow x \end{matrix}$  Basic Model

Inelastic Events Legend

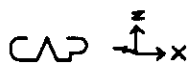
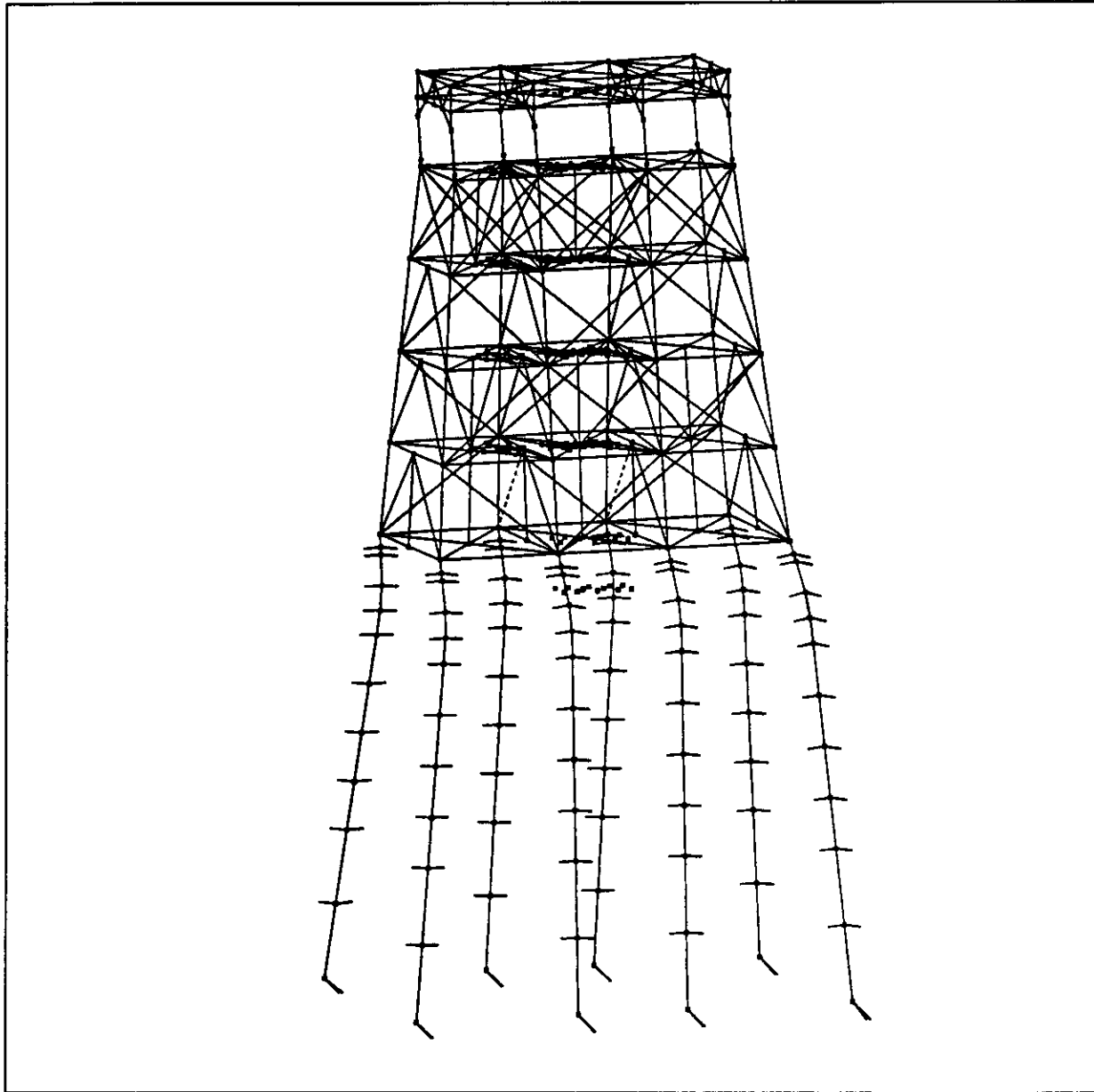
\_\_\_\_\_ Elastic  
 ..... Strut Residual  
 ..... Plastic Strut/NLTruss  
 \_\_\_\_\_ Beam Clmn Fully Plastic

..... Strut Buckling  
 \_\_\_\_\_ Strut Reloading  
 ..... Beam Clmn Initial Yield

**Figure 5-4 Basic Model: Last Load Step**



**Figure 5-5 Basic Model Results**



Soils Improved 100%

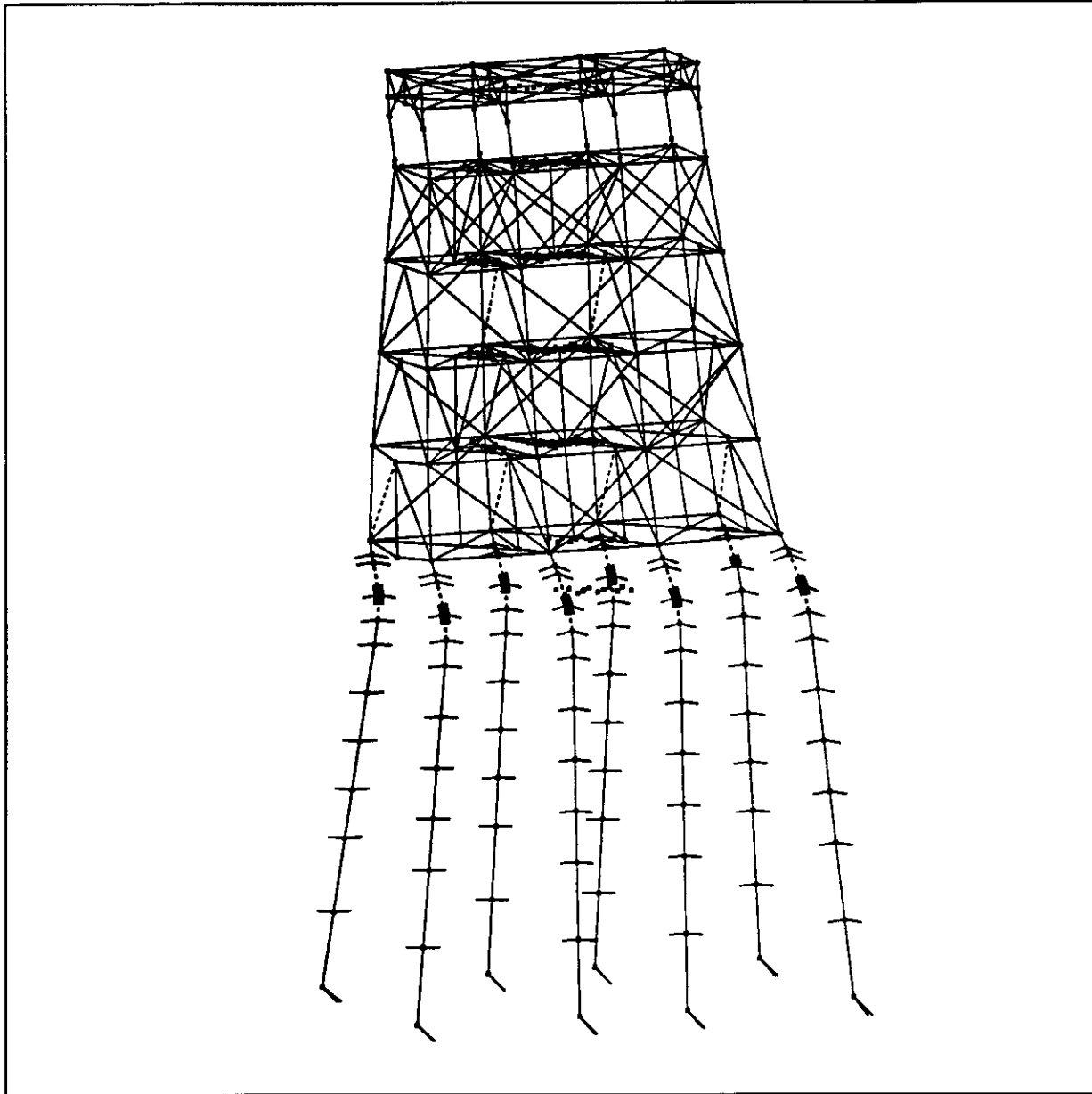
Inelastic Events Legend

— Elastic  
 - - - - - Strut Residual  
 . . . . . Plastic Strut/NLTruss  
 — Beam Cln Fully Plastic

- - - - - Strut Buckling  
 — Strut Reloading  
 . . . . . Beam Cln Initial Yield

**Figure 5-6 Increased Foundation: Load Step at First Yield**



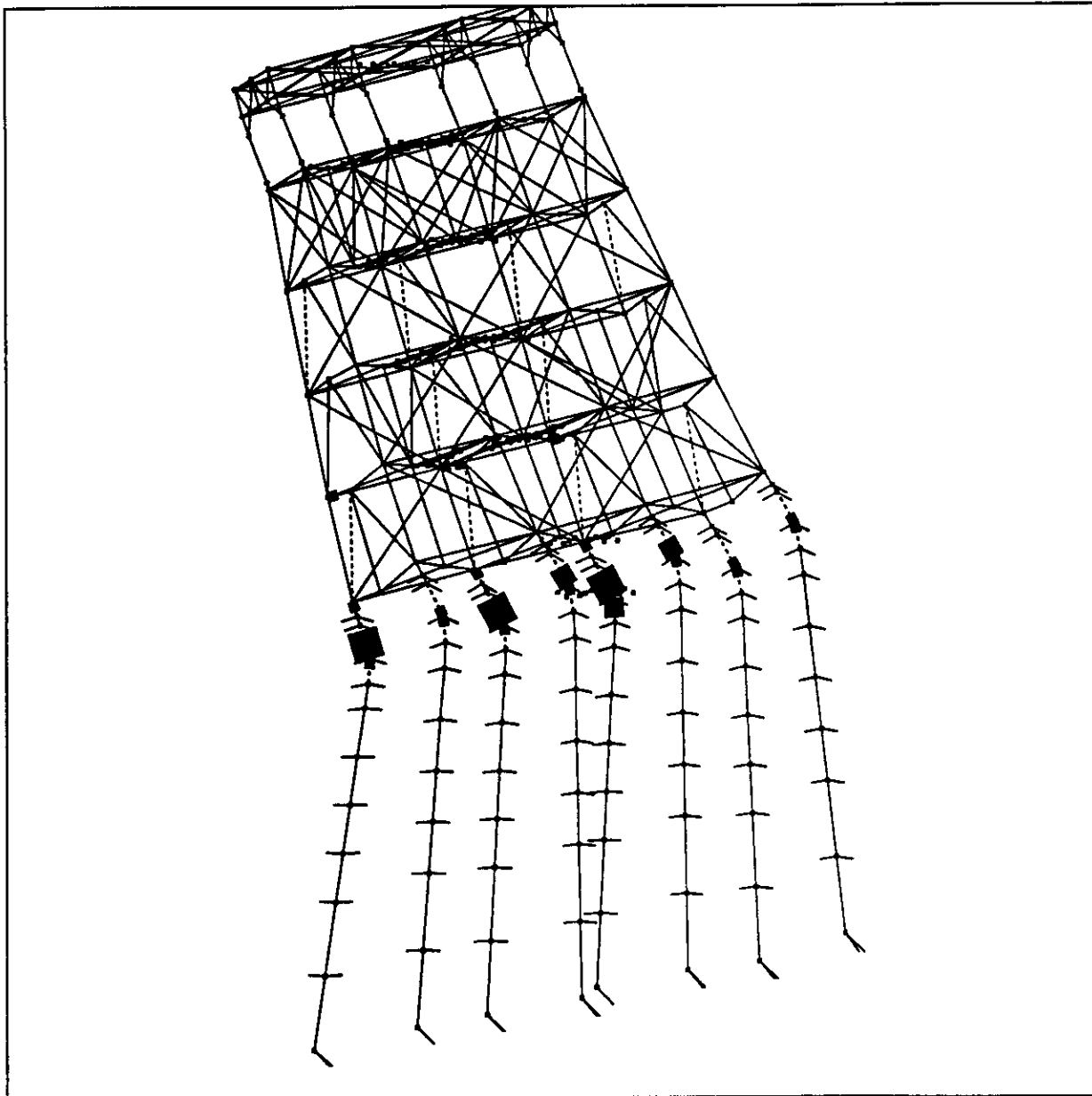


CAP  $\begin{matrix} \uparrow z \\ \rightarrow x \end{matrix}$  Soils Improved 100%

Inelastic Events Legend

————	Elastic	-----	Strut Buckling
.....	Strut Residual	————	Strut Reloading
.....	Plastic Strut/NLTruss	.....	Beam Clmn Initial Yield
————	Beam Clmn Fully Plastic		

**Figure 5-7 Increased Foundation: Intermediate Load Step**



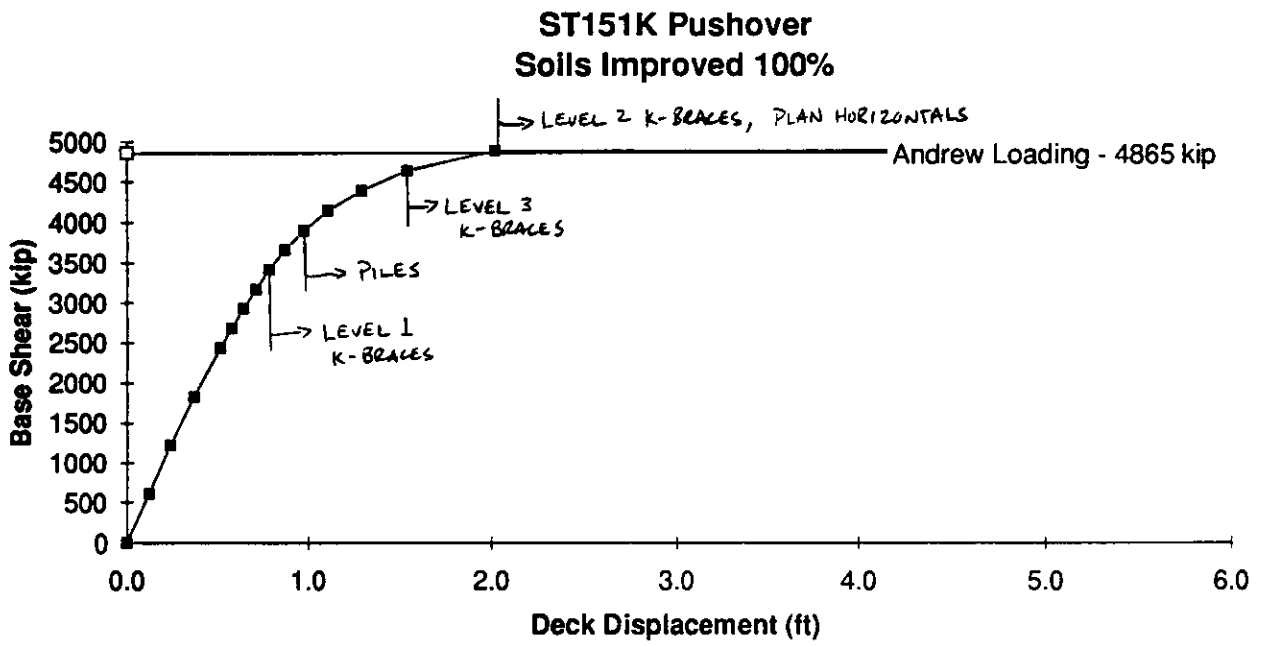
CAP  $\uparrow$   $\rightarrow$  x Soils Improved 100%

Inelastic Events Legend

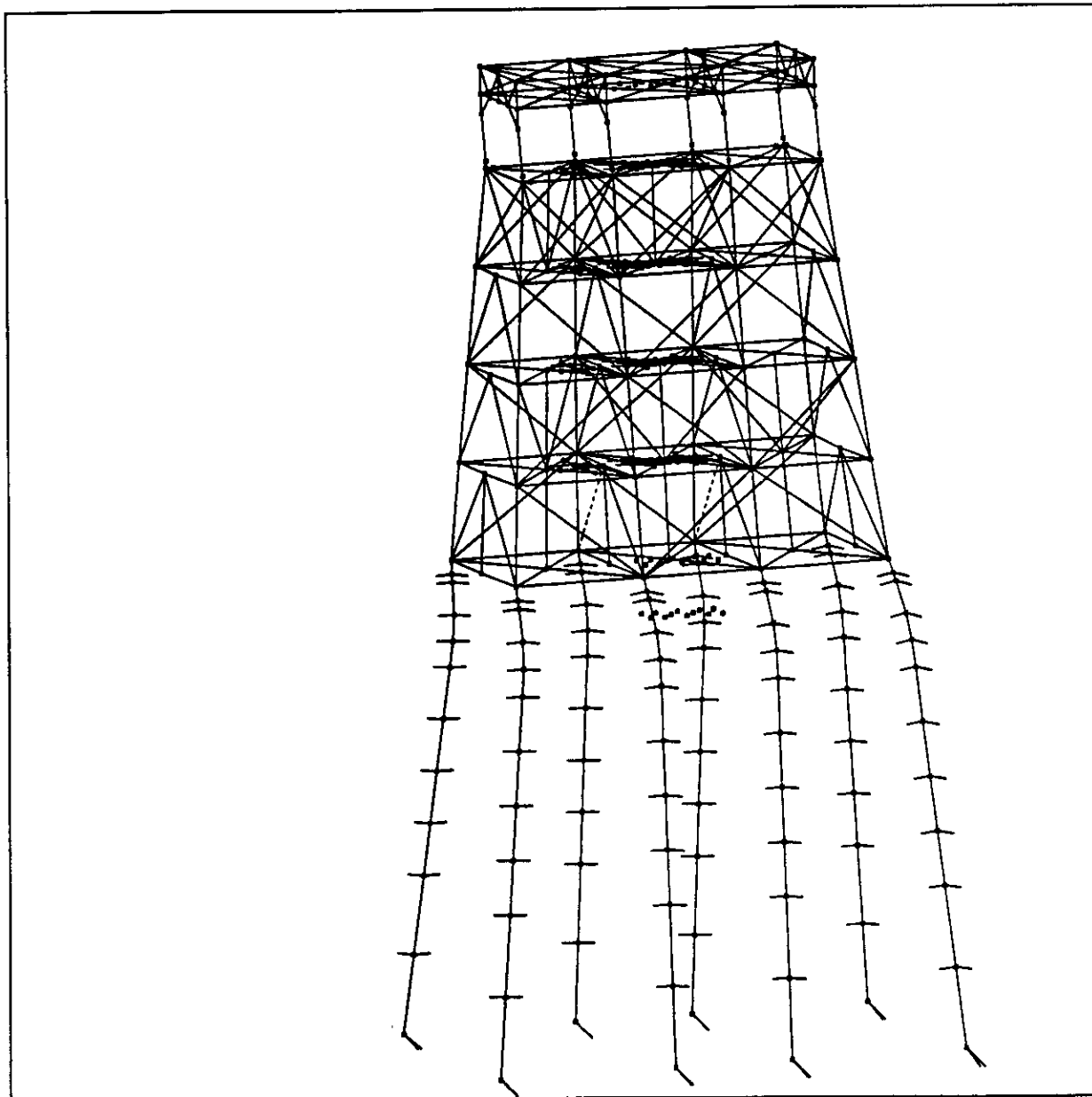
\_\_\_\_\_ Elastic  
 ..... Strut Residual  
 ..... Plastic Strut/NLTruss  
 \_\_\_\_\_ Beam Clmn Fully Plastic

..... Strut Buckling  
 \_\_\_\_\_ Strut Reloading  
 ..... Beam Clmn Initial Yield

**Figure 5-8 Increased Foundation: Last Load Step**



**Figure 5-9 Increased Foundation Results**

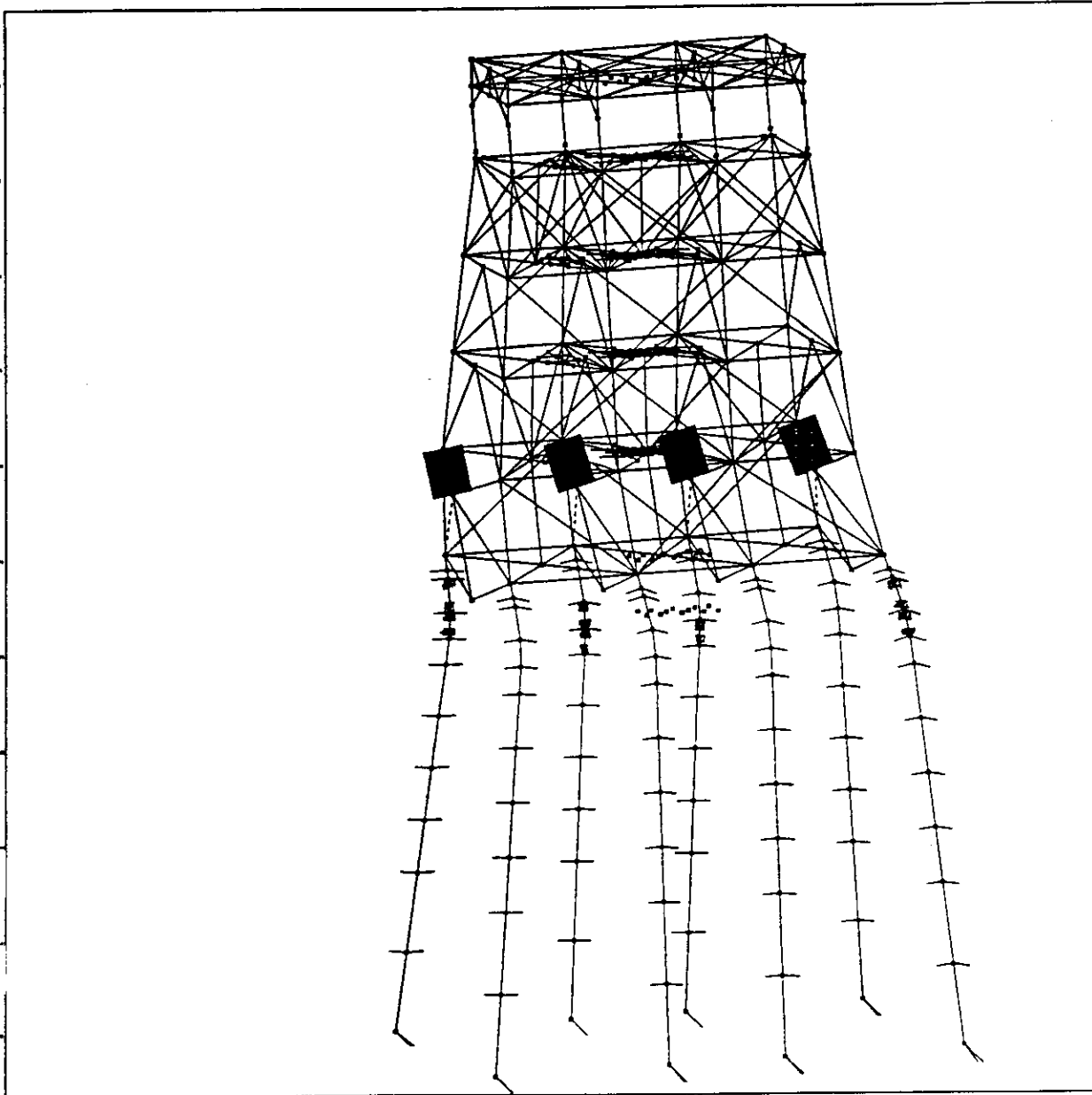


Soils Improved 100%, Brittle K-Joints

Inelastic Events Legend

————	Elastic	-----	Strut Buckling
.....	Strut Residual	————	Strut Reloading
.....	Plastic Strut/NLTruss	.....	Beam Clmn Initial Yield
————	Beam Clmn Fully Plastic		

**Figure 5-10 Brittle Joints: Load Step at First Yield**



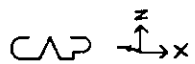
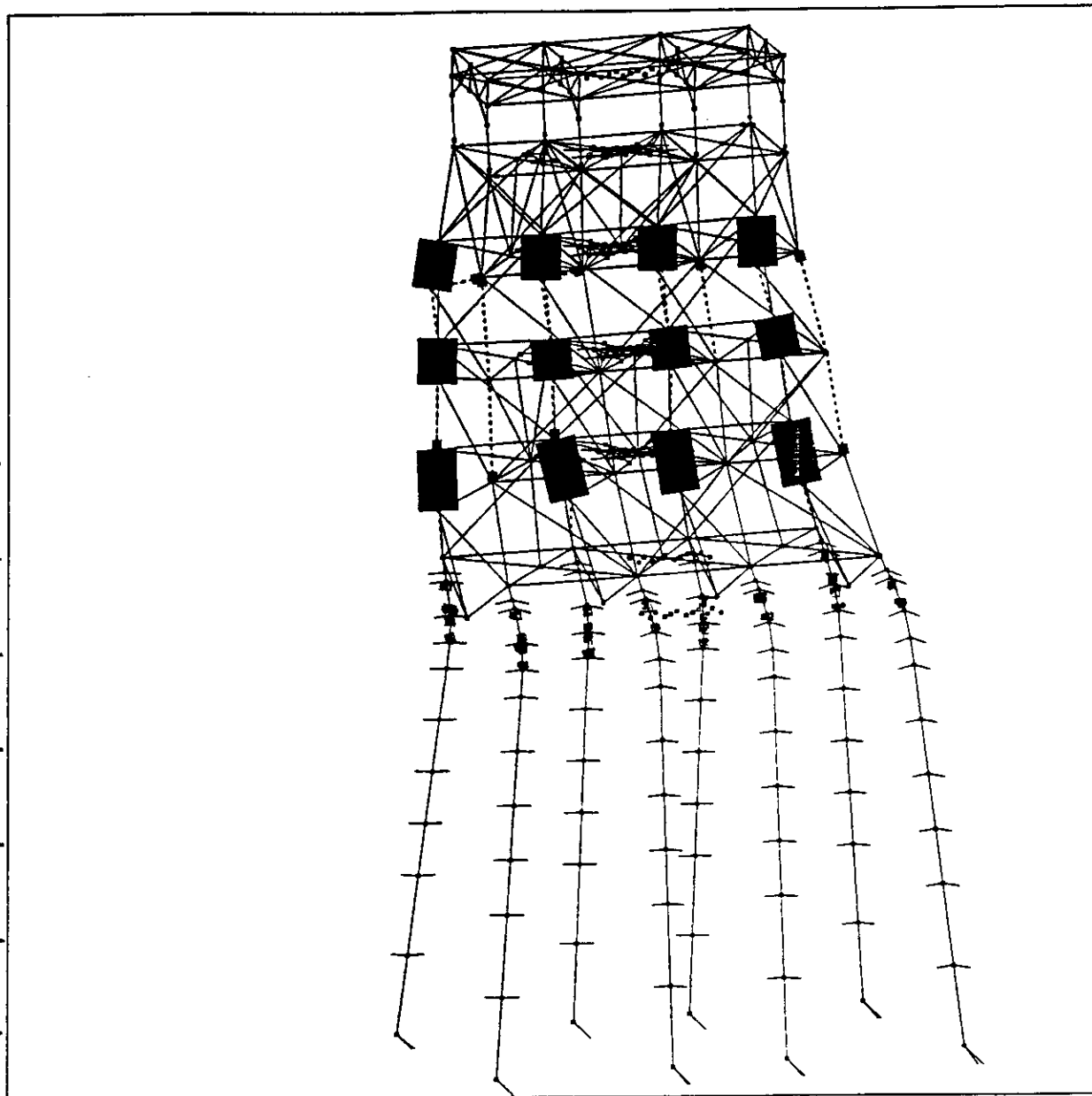
CAP  $\begin{matrix} \uparrow z \\ \rightarrow x \end{matrix}$

Soils Improved 100%, Brittle K-Joints

Inelastic Events Legend

—————	Elastic	-----	Strut Buckling
-----	Strut Residual	—————	Strut Reloading
.....	Plastic Strut/NLTruss	.....	Beam Clmn Initial Yield
—————	Beam Clmn Fully Plastic		

**Figure 5-11 Brittle Joints: Intermediate Load Step**

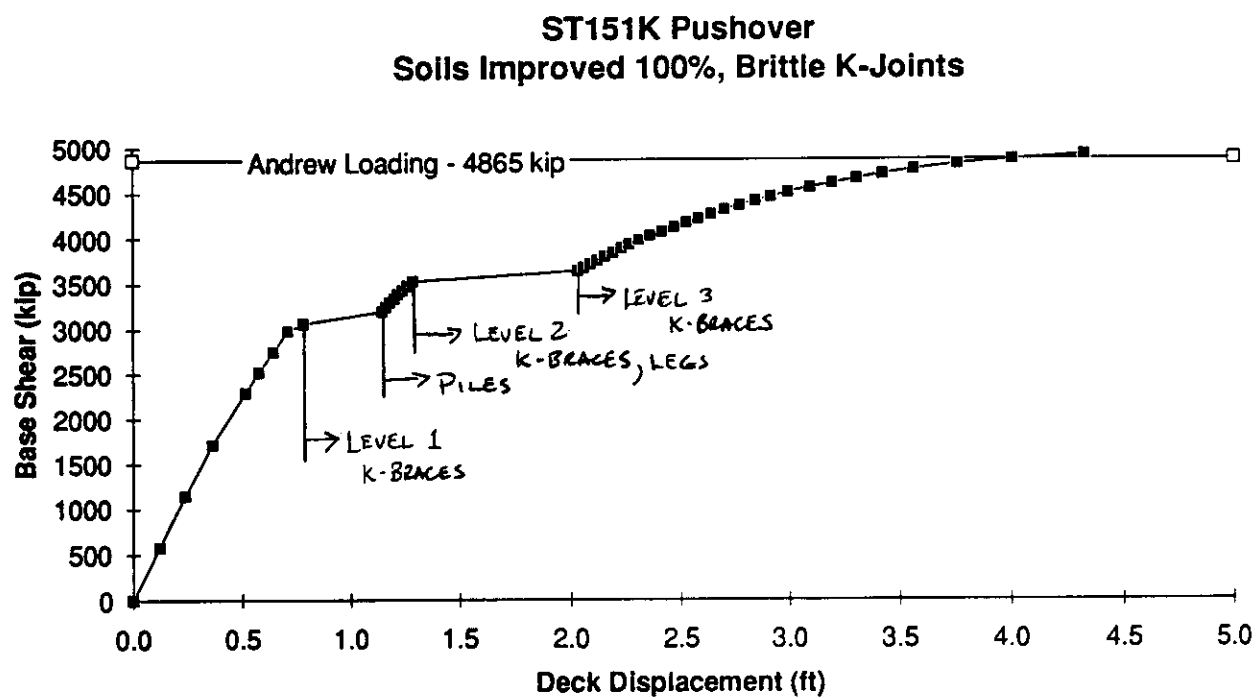


Soils Improved 100%, Brittle K-Joints

Inelastic Events Legend

—————	Elastic	-----	Strut Buckling
.....	Strut Residual	—————	Strut Reloading
.....	Plastic Strut/NLTruss	.....	Beam Clmn Initial Yield
—————	Beam Clmn Fully Plastic		

**Figure 5-12 Brittle Joints: Last Load Step**



**Figure 5-13 Brittle Joints: Results**

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## SECTION 6            ST 130 Q ANALYSIS

### 6.1    INTRODUCTION

A CAP model was created of the ST-130 Q structure, based on drawings provided by Chevron for the Andrew JIP. The platform has no conductors and no boat landing. Pushover analyses were performed using environmental loads based on the most severe hour of Andrew. A set of analyses of increasing sophistication were performed and their results compared in order to determine the effect of analysis assumptions.

The following analyses were performed:

- Basic Model  
  
Soil strength per API RP 2A, elasto-plastic joint failure.
- Increased Foundation Strength Model (50%)  
  
Same as the basic model, except that the soil shear strength is increased by 50%. See Section 4.1 for discussion regarding strengthening of the foundation.
- Increased Foundation Strength Model (100%)  
  
Same as the previous case except that the soil strength is improved by 100% (doubled).
- Soil Nodes Fixed  
  
All nodes below the mudline fixed to determine the effect of the foundation on the platform capacity.
- Brittle K-Joints  
  
Same as the basic model, except that the chord and brace elements of the K-joints are modeled to fail in brittle fashion.
- Brittle K-Joints - Increase Foundation (200%)  
  
Same as the basic model, except that there is a 200% increase soil strength, and the chord and brace elements of the K-joints are modeled to fail in a brittle fashion.

## **6.2. DESCRIPTION OF MODEL**

A plot of the model is shown in Figure 6-1.

### **6.2.1 Substructure**

The substructure was modeled per the provided Chevron drawings, with the following exceptions and comments:

- Node cans were neglected (except for computing joint strength)
- Risers and caissons were neglected
- The boat landing was removed based upon a survey of the platform during the inspection described in Section 2 (in the previous Andrew JIP, the boat landing had been included in the model).

### **6.2.2 Superstructure**

Primary members in the superstructure were modeled explicitly. The deck was modeled approximately with the effective member sizes chosen to reflect the estimated value of deck stiffness and weight. The deck weight was 1480 kips.

### **6.2.3 Conductors**

There are no conductors.

### **6.2.4 Piles and Soil**

The piles are 30 inches in diameter and have a penetration of 240 feet.

Soil properties for this analysis model were based on the soil properties at the ST 151 K site.

### **6.2.5 Element Types**

The element types used in the model were the same as that described for the ST 130 A platform in Section 4.2.5.

## **6.3 ENVIRONMENTAL LOADS**

## MMS Platform Inspections and Analysis

The environmental loads were based on Andrew data for the worst hour of the storm, as follows:

- Maximum wave height = 62.3 ft
- Associated peak spectral period = 14.2 sec
- Associated current velocity = 3.0 ft/sec
- Maximum wind speed = 135 ft/s
- Environmental direction = 289 deg off true north, or 314 deg from platform north
- Marine growth profile = 3 in. on diameter from mean low water to mudline

The following assumptions were made, based on API RP2A, 20th edition:

- Storm tide and surge = 2.0 ft
- Current profile is constant
- Current blockage factor = 0.85 (for diagonal direction)
- Wave spreading factor = 0.88
- Drag coefficient = 1.05 (0.65 for smooth members), inertia coefficient = 1.2 (1.6 for smooth members)

The wave reaches approximately five feet six inches into the subcellar deck. Using API Preliminary Deck Force Guidelines (API, 1994) the calculated wave-in-deck load is 230 kip and is divided evenly at the cellar deck level among the four legs.

Wind load on the deck was calculated using API RP2A (20th) recommendations. The wind load is more substantial on this platform compared to ST 130 A and ST 151 K due to the presence of the quarters building. The total wind force is 175 kips.

The combined wave and current load generated by CAP based on the above assumptions is 1070 kips. The majority is caused by the wave. Therefore, total environmental loading on the structure is 1475 kips.

### **6.4. MEMBER AND JOINT MODELING**

Member and joint capacities were computed using the same approach as described for ST 130 A. Details are provided in Section 4.4.

### 6.5 ANALYSIS RESULTS

#### 6.5.1 General

A summary of capacities for the various analyses, in the form of pushover load ratio at failure, is presented in Table 6-1. A summary plot of the pushover load ratio as a function of deck SRSS lateral displacement for all major analyses is shown in Figure 6-2.

Analysis results presented in the following sections include inelastic events and displaced shape plots. The displacement sin these plots have been scaled by a factor of 20 to more clearly see the platform deformations.

#### 6.5.2 Basic Model

The capacity of the basic model is 1.12 times the Andrew environmental pushover load. The displaced shape and first yield are shown in Figure 6-3. Yielded and failed elements with the corresponding displaced shape for the last step is shown in Figure 6-4. The analysis terminated due to a large unbalance force in the northwest pile, but it is clear that global failure would be caused by lateral pile failure.

The pushover load plot is shown in Figure 6-5, with major events indicated. The general behavior is a gradually, fairly smooth, decreasing stiffness, with no abrupt changes in behavior until the pile failure. The first elements to fail are in the northwest corner pile. This occurs at a load level of 0.96 times the total environmental load. The southeast corner pile begins to yield at a load ratio of 1.05. None of the K-braces fail in this analysis.

#### 6.5.3 Increased Foundation Strength Model (50%)

The capacity of the model with soils improved 50% is 1.38 times the Hurricane Andrew environmental pushover load. The displaced shape and first yield are shown in Figure 6-6. Yielded elements and the corresponding displaced shape for the last load step are shown in Figure 6-7. The analysis terminated successfully when the maximum allowable displacement of five feet was reached. Leg elements below

The pushover load plot is shown in Figure 6-8, with major events indicated. The northwest pile begins to yield at the same load ratio of 0.96 as the previous case, and the southeast corner pile begins to yield at a load ratio of 1.14. No brace elements fail.

#### **6.5.4 Increased Foundation Strength Model (100%)**

The capacity of the model with soils improved 100% is 1.58 times the Andrew environmental pushover load or about 40% greater than the basic model's capacity. The displaced shape and first yield are shown in Figure 6-9. Yielded elements with the corresponding displaced shape for the last load step are shown in Figure 6-10. Note that for this analysis, one of the K-braces has failed (dashed line in Figure 6-10), indicating that the increase in foundation strength is forcing member failures into the jacket.

The pushover load plot is shown in Figure 6-11, with major events indicated. The general behavior is that of a gradually, smoothly decreasing stiffness, with no abrupt changes in behavior until the piles fail completely. The diagonal brace does not fail until the foundation has completely yielded and the failure mechanism is fully developed. First yield occurs in the northwest pile at a load ratio of 0.96 times the Andrew load. The southeast pile begins to yield at a load ratio of 1.14 times the Andrew load. Compared to the basic model, the 100% increase in soil strength does again not increase the load level of initial yield.

### **6.5.5 Soil Nodes Fixed**

The capacity of the model with the soil nodes fixed is 2.89 times the Hurricane Andrew environmental pushover load or about 260% greater than the basic model's capacity. This analysis demonstrates the effect of the foundation on the platform's capacity. The displaced shape and first yield are shown in Figure 6-12. Yielded and failed elements and their corresponding displaced shapes for an intermediate load step and the last load step are shown in Figures 6-13 and 6-14. A majority of the K joints have failed, with the platform failure mode occurring in the legs just above the mudline.

The pushover load plot is shown in Figure 6-15, with major events indicated. The legs begin to yield at a load ratio of 1.77 times the environmental load. The northwest leg fails at a load ratio of 2.38 times the environmental load. The compression K-braces begin to fail at a load ratio of 1.84 and continue to fail through the last load step. The horizontals begin to yield at a load ratio of 2.35, one load step before the northwest leg fails.

### **6.5.6 Basic Model with SeaStar Element 14, Brittle K-Joints**

The capacity of the basic model with brittle K-joints, is 1.13 times the Hurricane Andrew environmental pushover load or about equal to the basic model's capacity. The apparent increase in the capacity (about 1%) of this analysis can be attributed to the size of the load step. The displaced shape and first yield are shown in Figure 6-16. Yielded and failed elements and their corresponding displaced shape for the last load step are shown in Figure 6-17. The analysis terminated due to large displacements, with the failure mechanism still in the piles.

The pushover load plot is shown in Figure 6-18, with major events indicated. The northwest pile begins to yield at a load ratio of 0.96 and the southeast corner pile begins to yield at a load ratio of 1.05. None of the K-joints fail in this analysis as would be expected, since none of the K-joints had failed in the previous analysis in which the soil strength was unchanged.

### **6.5.8 Soil Improved 200%, Brittle K-Joints**

The capacity of the model with soils improved 200% is 1.72 times the Hurricane Andrew environmental pushover load or about 54% greater than the basic model's capacity. The displaced shape and first yield are shown in Figure 6-19. Yielded elements with the corresponding displaced shape for the last load step are shown in Figure 6-20.

The pushover load plot is shown in Figure 6-21, with major events indicated. Similar to the previous analyses, the piles begin to yield much earlier than the joints. First yield occurs in the

northwest pile at a load ratio of 1.07 times the Andrew load. The southeast pile begins to yield at a load ratio of 1.25 times the Andrew load and the northwest and southeast legs below the mudline yield at a load ratio of 1.60 times the Andrew load. Global failure occurs at a load ratio of 1.43 times the Andrew load. At the point of global failure the load increment had been reduced to 0.1% of the Andrew load (approximately 1 kip) to reduce the amount of load introduced into the system and control the unbalanced forces.

The SE pile begins to yield at a load ratio of 1.25 times the Andrew load. Compared to the basic model, the 50% soil improvement, and the 100% soil improvement, the 200% increase in soil strength increases the capacity of the piles.

### **6.6 Conclusions**

The structure did not experience any observable failures due to Andrew. The analyses performed on the various models of the structure all indicate that the ultimate capacity of the structure is greater than the Andrew environmental loading and that the platform should not have collapsed, although the models with weaker soil shear strengths did predict that there should have at least been some damage to the foundation.

Soil strength is one of the controlling factors in the pushover analyses. The soil strengths were increased from 50 to 200 percent for this platform, with the 200 percent increase being the first case where no damage either to the jacket or to the foundation would be predicted for Andrew. This may imply that the required increase in foundation strength may be higher than the 100 percent increase found to work for the ST 151 K and ST 177 B (PMB, 1994d) platforms.

However, there may be other explanations for the lack of damage for this platform. This first is wave-in-deck loads which are significant for this structure, and as described for the ST 151 K platform, the wave may not have impacted the platform over its entire length as assumed for the analysis. A second factor is that perhaps there was damage to the foundation in terms of lateral displacement to the platform. The analysis indicate that the lateral movement would on the order of 6-12 inches, which may not have been noticeable in the platform inspections.

**Summary of ST130Q Pushover Analysis**

<b>Run</b>	<b>Ultimate Capacity (kip)</b>	<b>Ratio to Andrew Loading</b>
<b>Basic Model with BC El. 14</b>	1650	1.12
<b>Soil Improved 50%</b>	2040	1.38
<b>Soil Improved 100%</b>	2335	1.58
<b>Soil Nodes Fixed</b>	4260	2.89
<b>Basic Model with Brittle K-Joints</b>	1665	1.13
<b>Soil Improved 200% with Brittle K-Joints</b>	2535	1.72

**Table 6-1 Summary of ST 130 Q Results**



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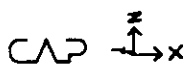
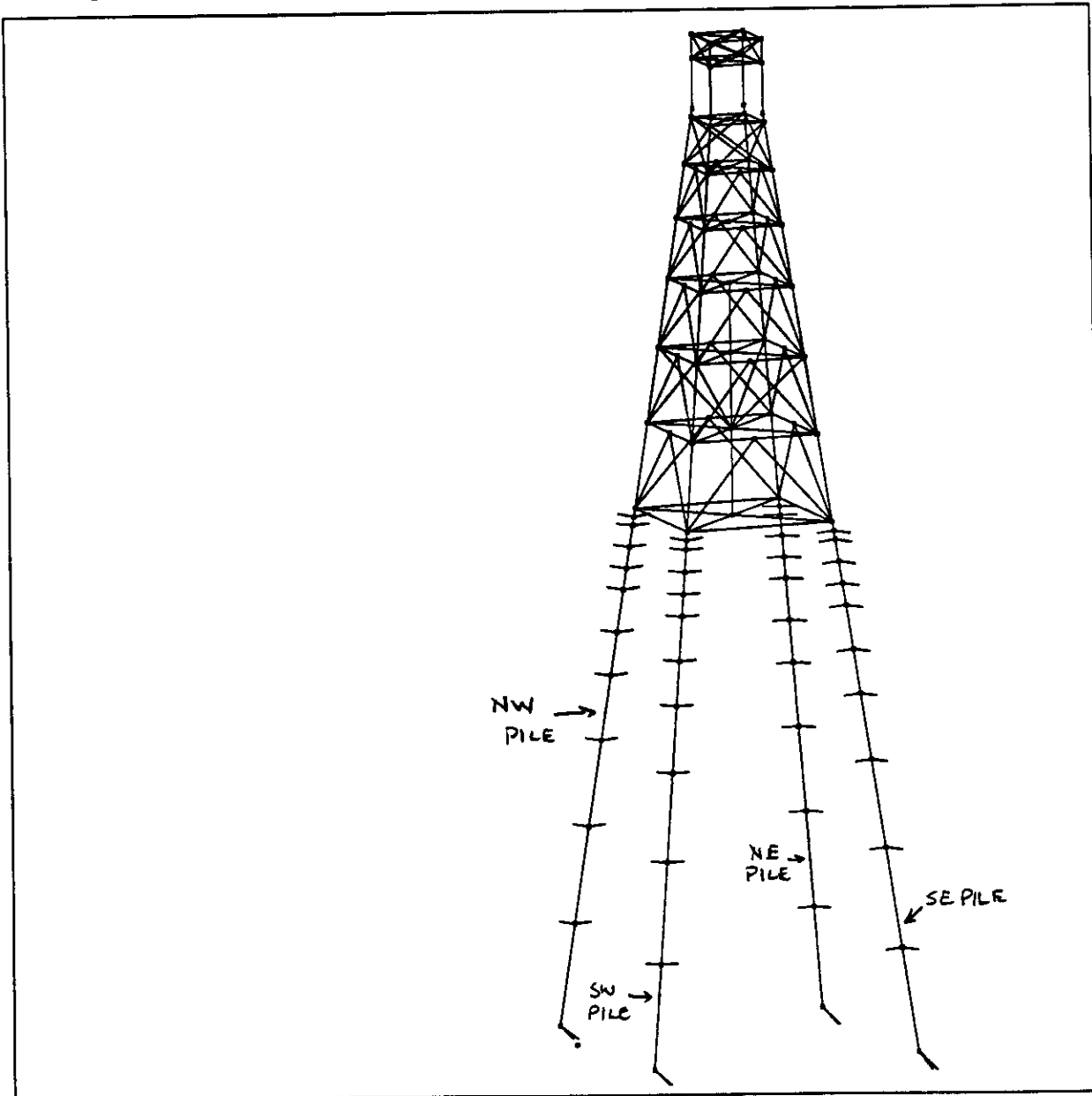
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Model View

**Figure 6-1 ST 130 Q CAP Computer Model**

# Comparison of Pushovers ST130Q - Diagonal Direction

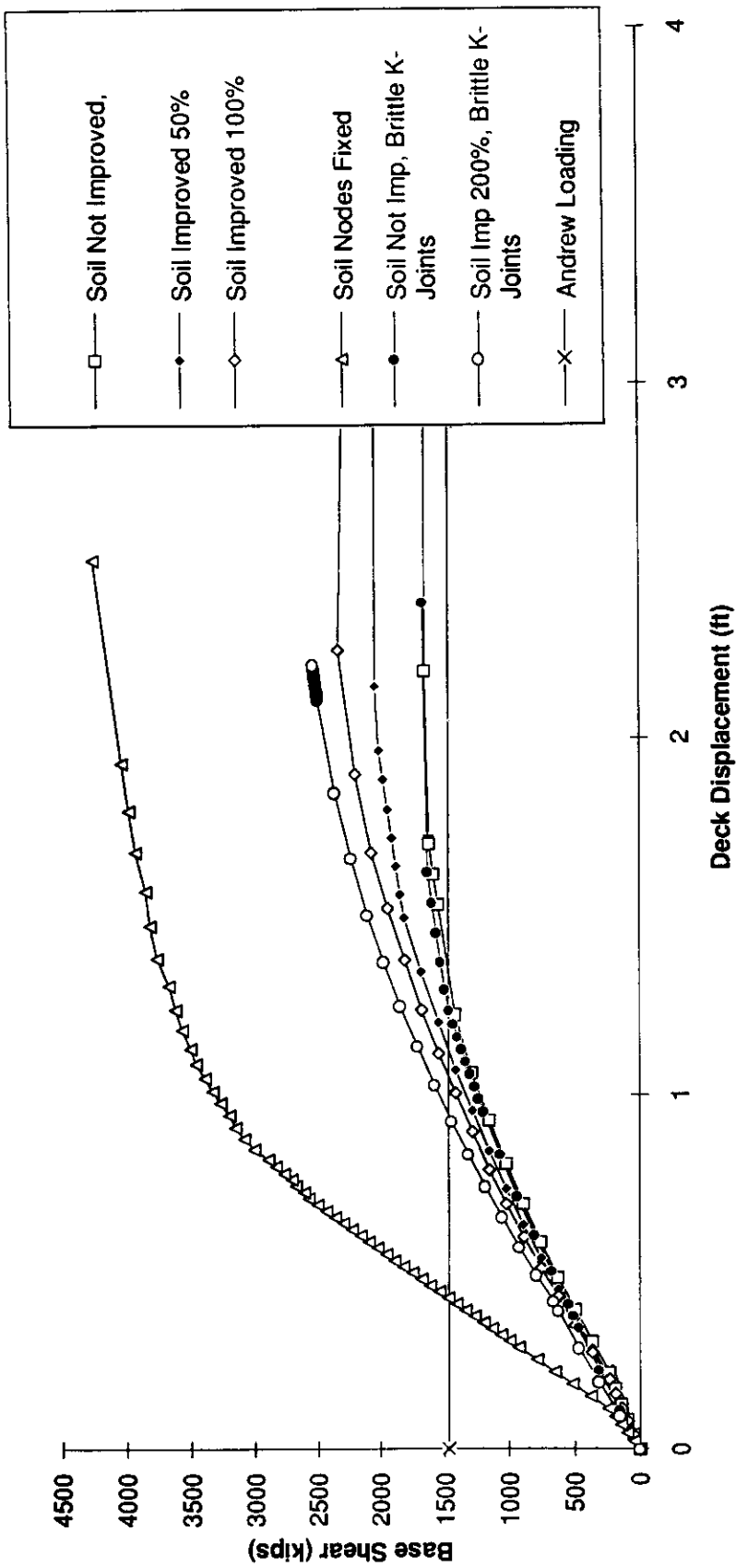
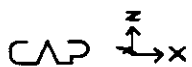
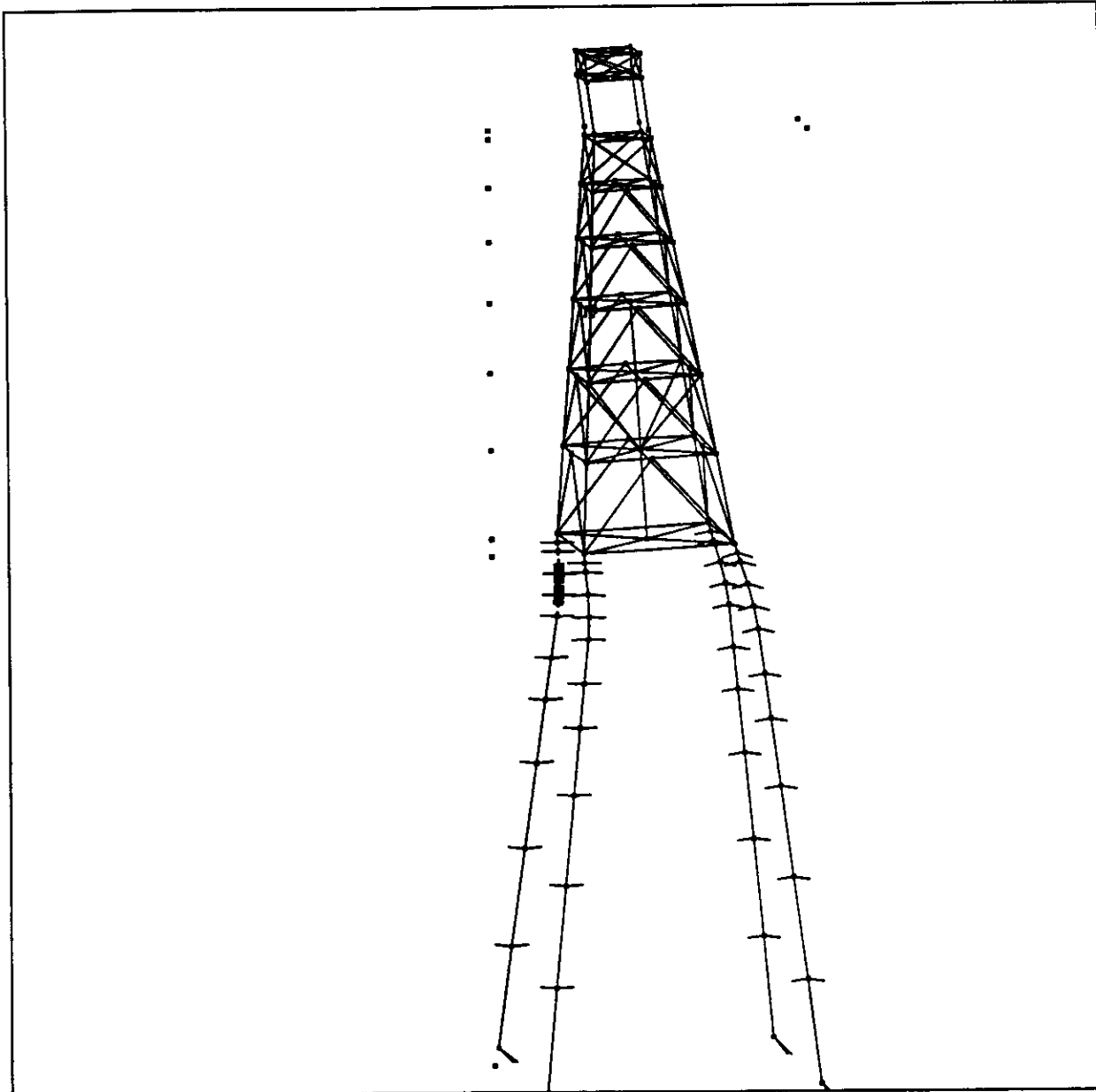


Figure 6-2 ST 130 Q Pushover Comparison of Analysis Results



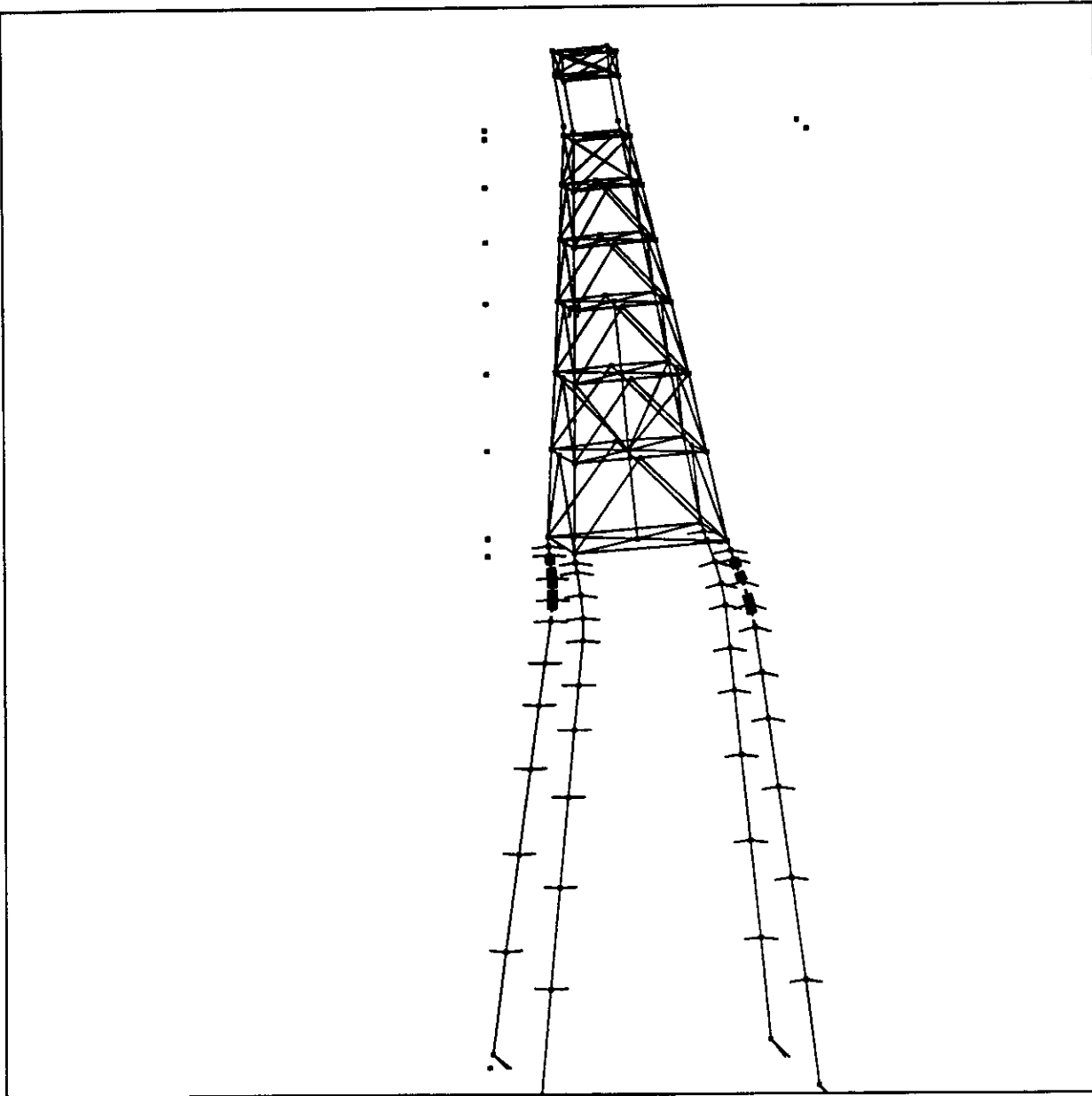
Basic Model Snapshot 1

Inelastic Events Legend

— Elastic  
 - - - - - Strut Residual  
 . . . . . Plastic Strut/NLTruss  
 — Beam Clmn Fully Plastic

- - - - - Strut Buckling  
 — Strut Reloading  
 . . . . . Beam Clmn Initial Yield

Figure 6-3 Basic Model: Load Step at First Yield

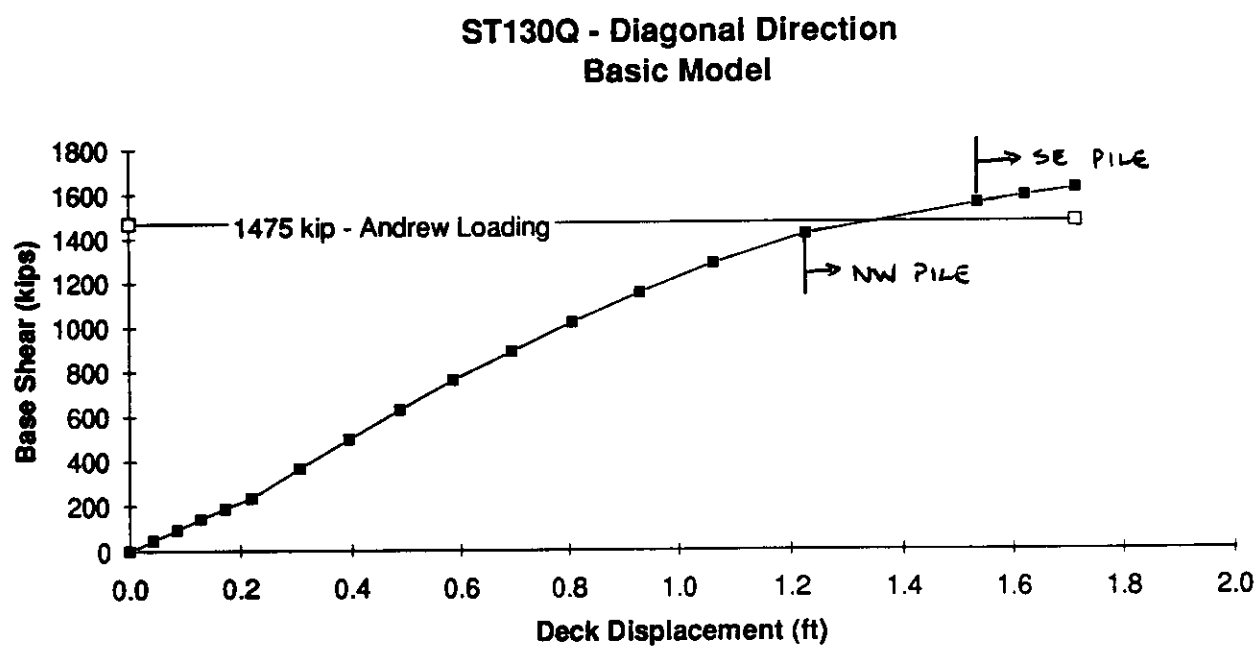


Basic Model Snapshot 2

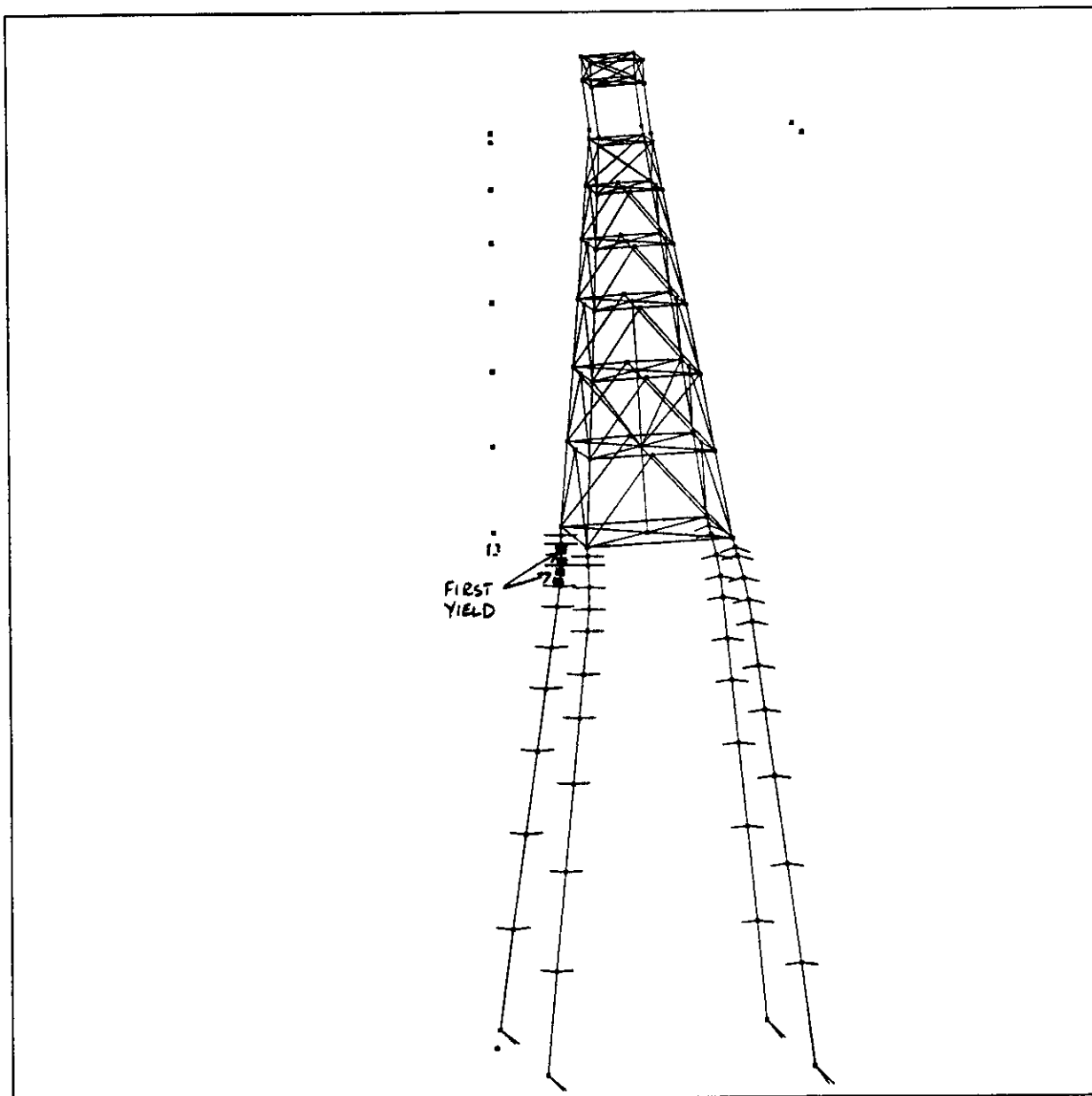
Inelastic Events Legend

————	Elastic	-----	Strut Buckling
-----	Strut Residual	————	Strut Reloading
.....	Plastic Strut/NLTruss	.....	Beam Clmn Initial Yield
=====	Beam Clmn Fully Plastic		

Figure 6-4 Basic Model: Last Load Step



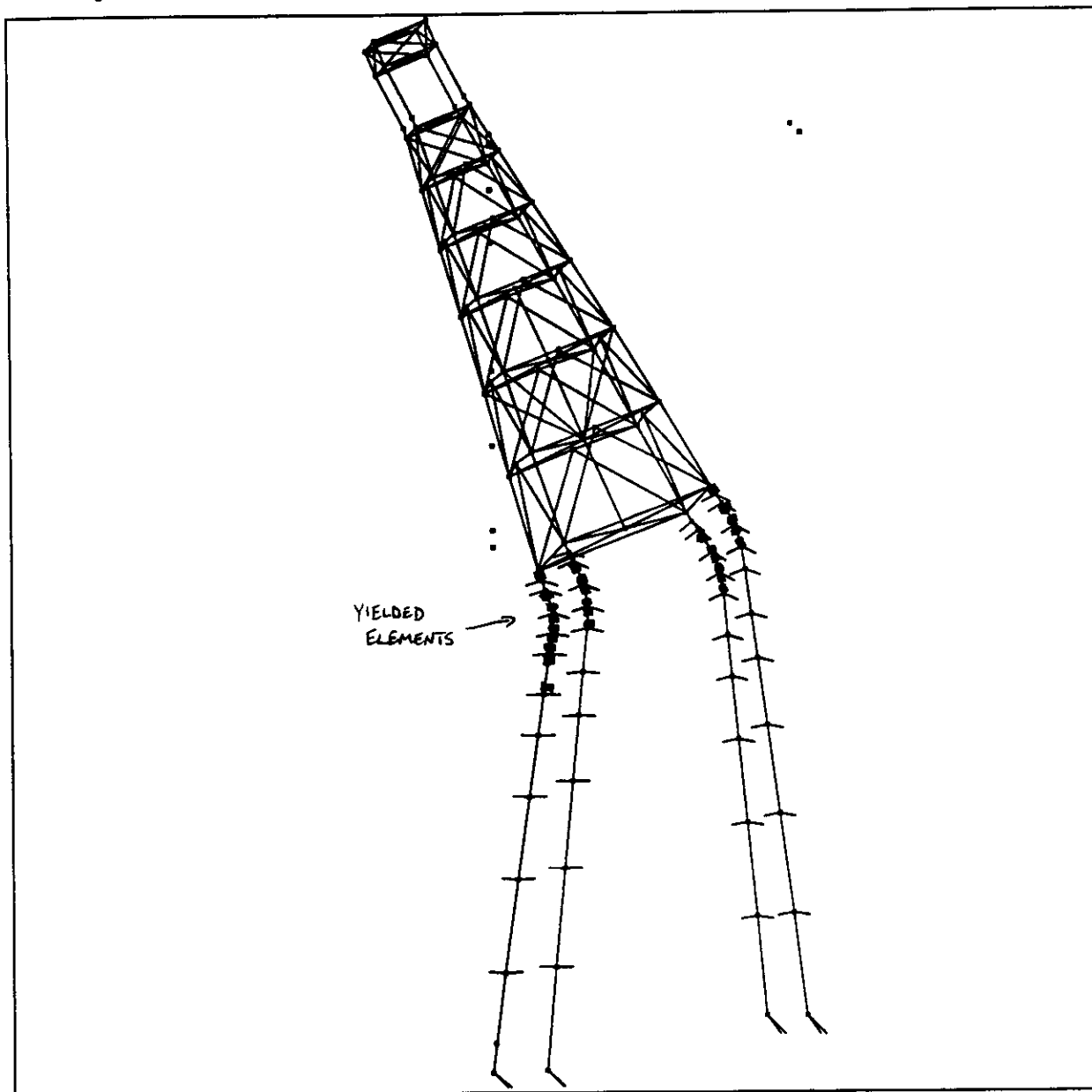
**Figure 6-5 Basic Model Results**



CAP  $\begin{matrix} \uparrow z \\ \rightarrow x \end{matrix}$

Soil Improved 50% Snapshot 1

**Figure 6-6 Increased Foundation (50%): Load Step at First Yield**



Soils Improved 50% Snapshot 2

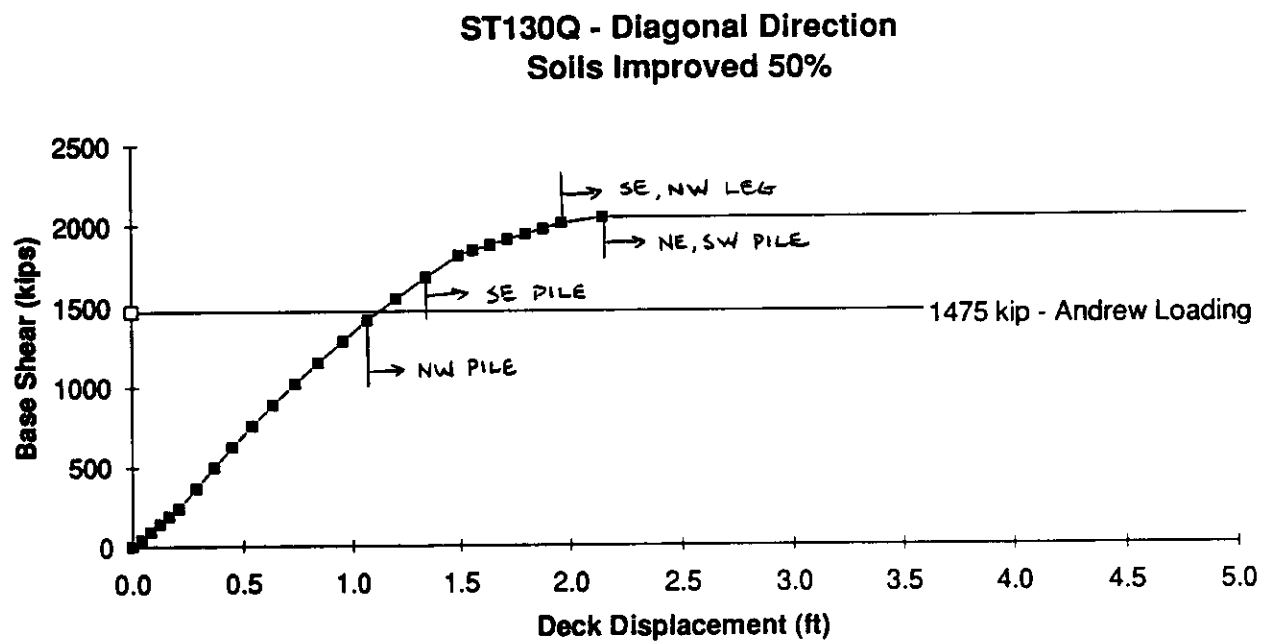
Inelastic Events Legend

\_\_\_\_\_ Elastic  
 - - - - - Strut Residual  
 . . . . . Plastic Strut/NLTruss  
 \_\_\_\_\_ Beam Clmn Fully Plastic

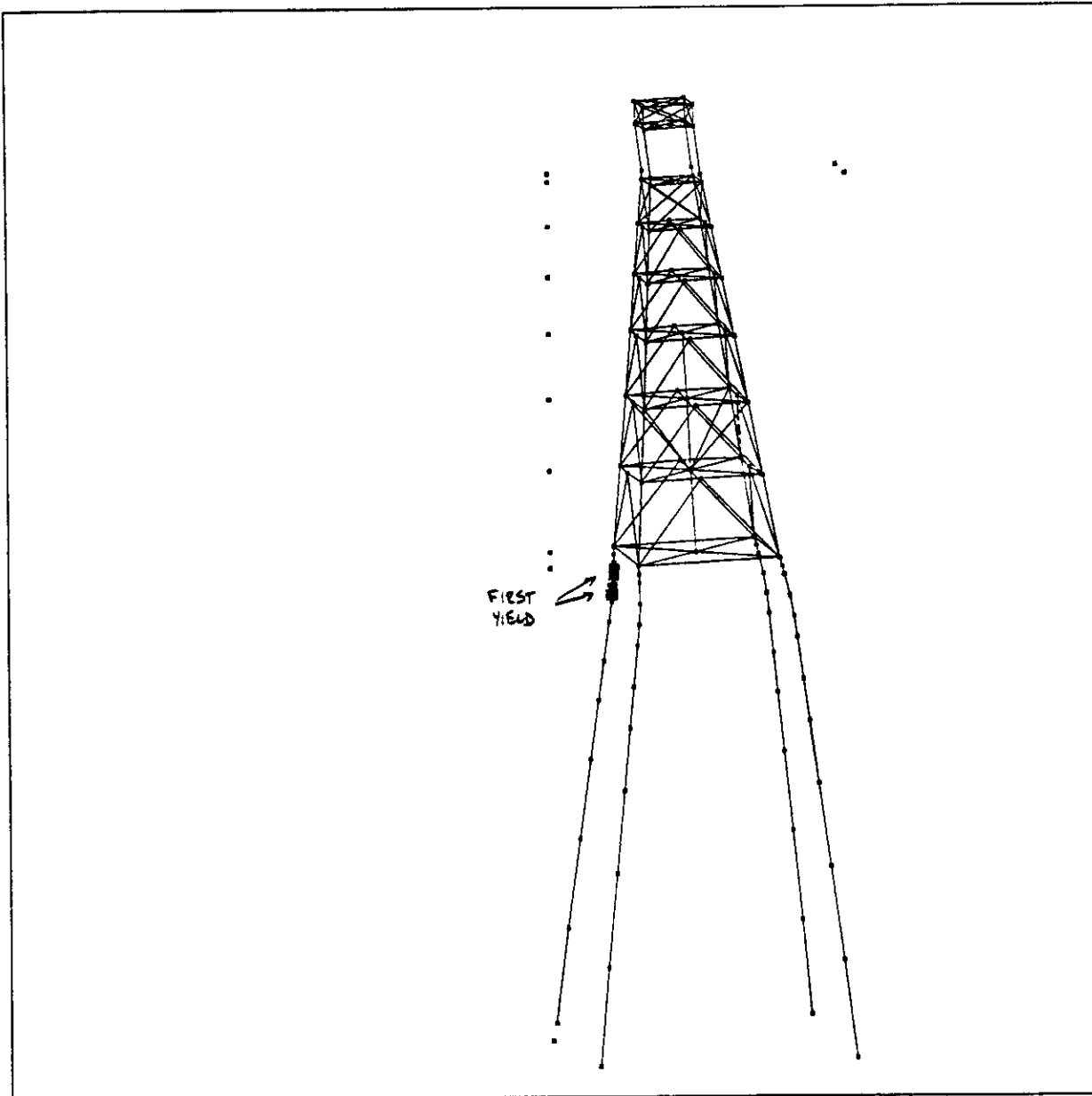
- - - - - Strut Buckling  
 \_\_\_\_\_ Strut Reloading  
 . . . . . Beam Clmn Initial Yield

Figure 6-7 Increased Foundation (50%): Last Load Step



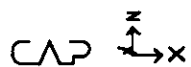
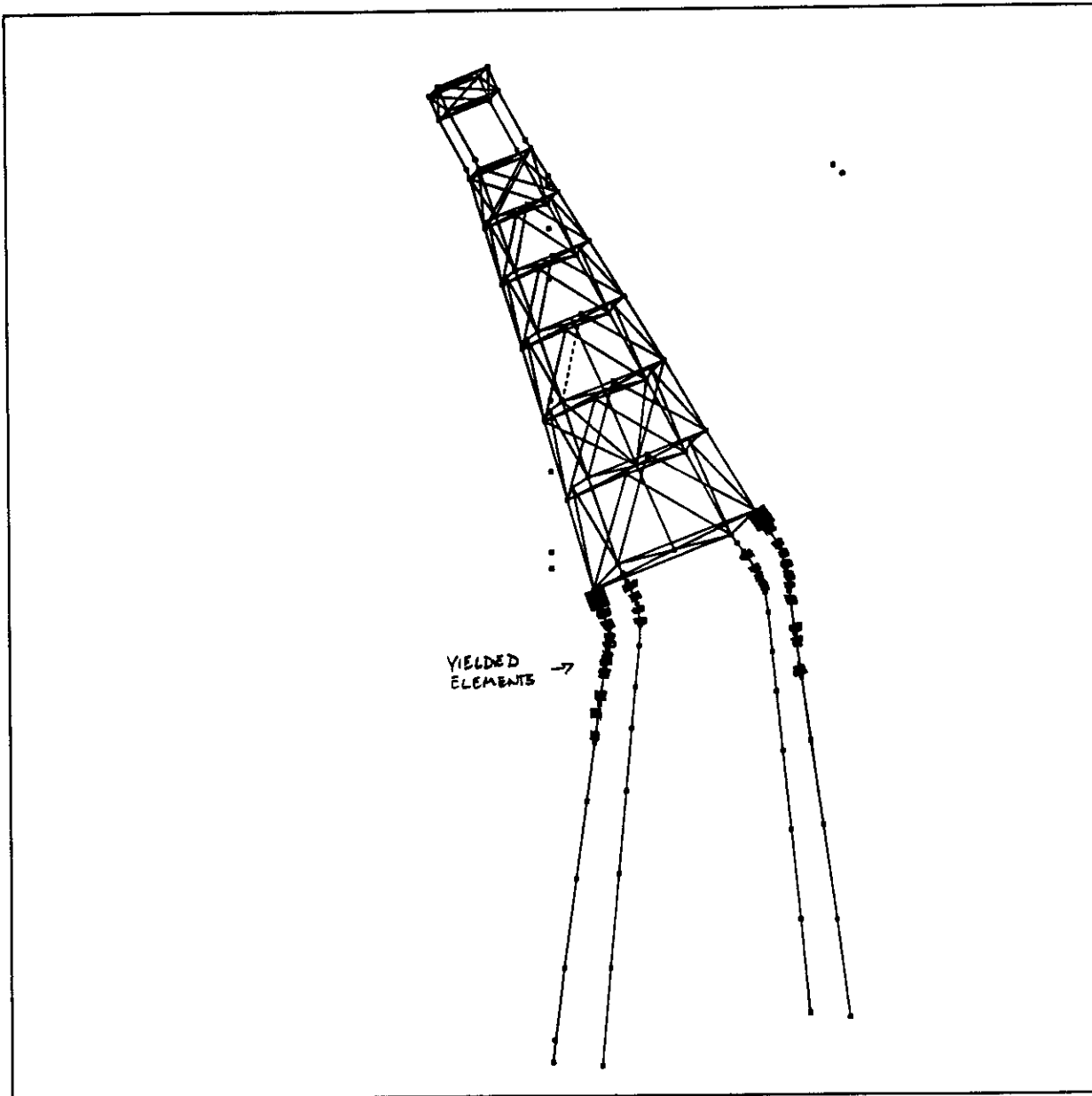


**Figure 6-8 Increased Foundation (50%) Results**



CLP  $\begin{matrix} \uparrow z \\ \rightarrow x \end{matrix}$  Soil Improved 100% Snapshot 1

**Figure 6-9 Increased Foundation (100%): Load Step at First Yield**



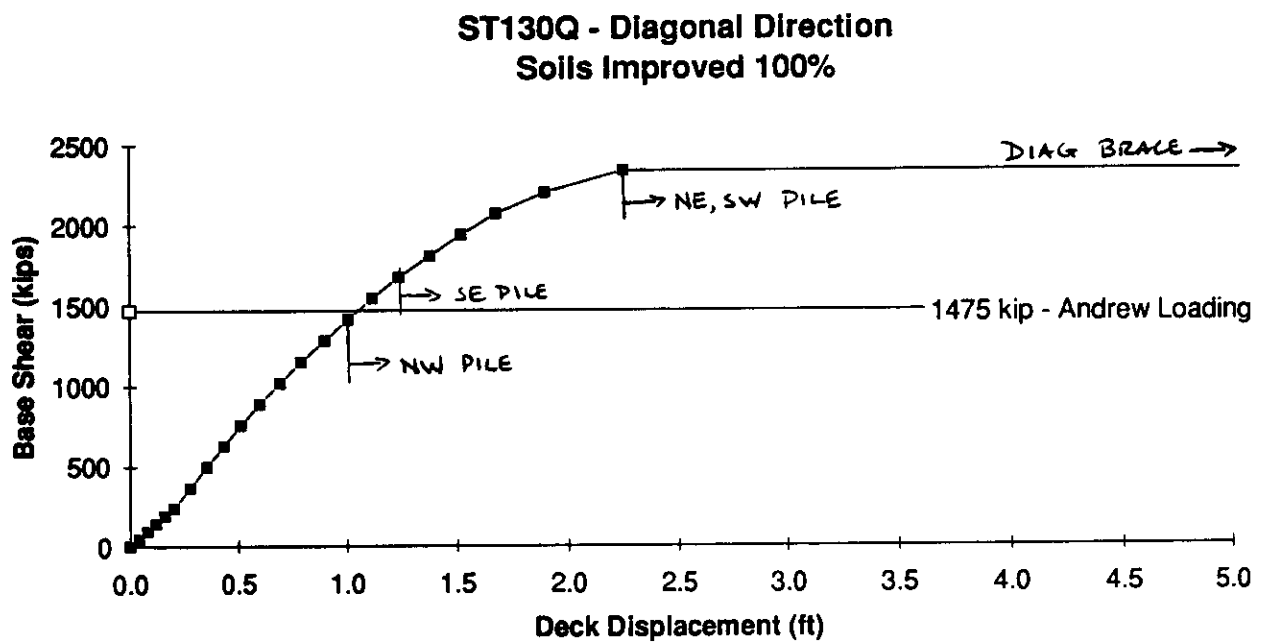
Soils Improved 100% Snapshot 2

Inelastic Events Legend

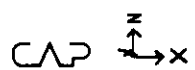
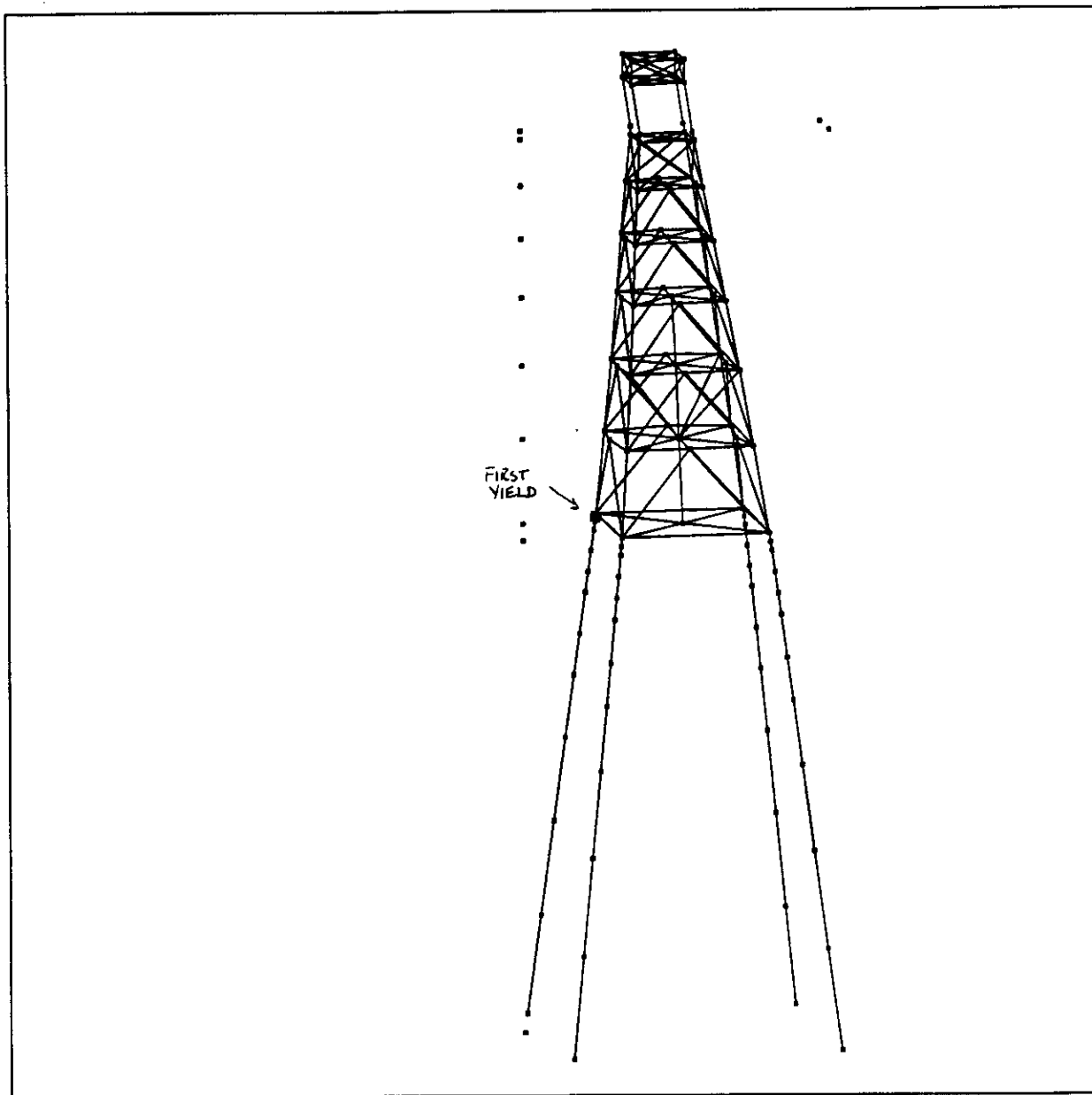
\_\_\_\_\_ Elastic  
 - - - - - Strut Residual  
 . . . . . Plastic Strut/NLTruss  
 \_\_\_\_\_ Beam Clmn Fully Plastic

- - - - - Strut Buckling  
 \_\_\_\_\_ Strut Reloading  
 . . . . . Beam Clmn Initial Yield

**Figure 6-10 Increased Foundation (100%): Last Load Step**



**Figure 6-11 Increased Foundation (100%) Results**



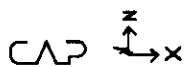
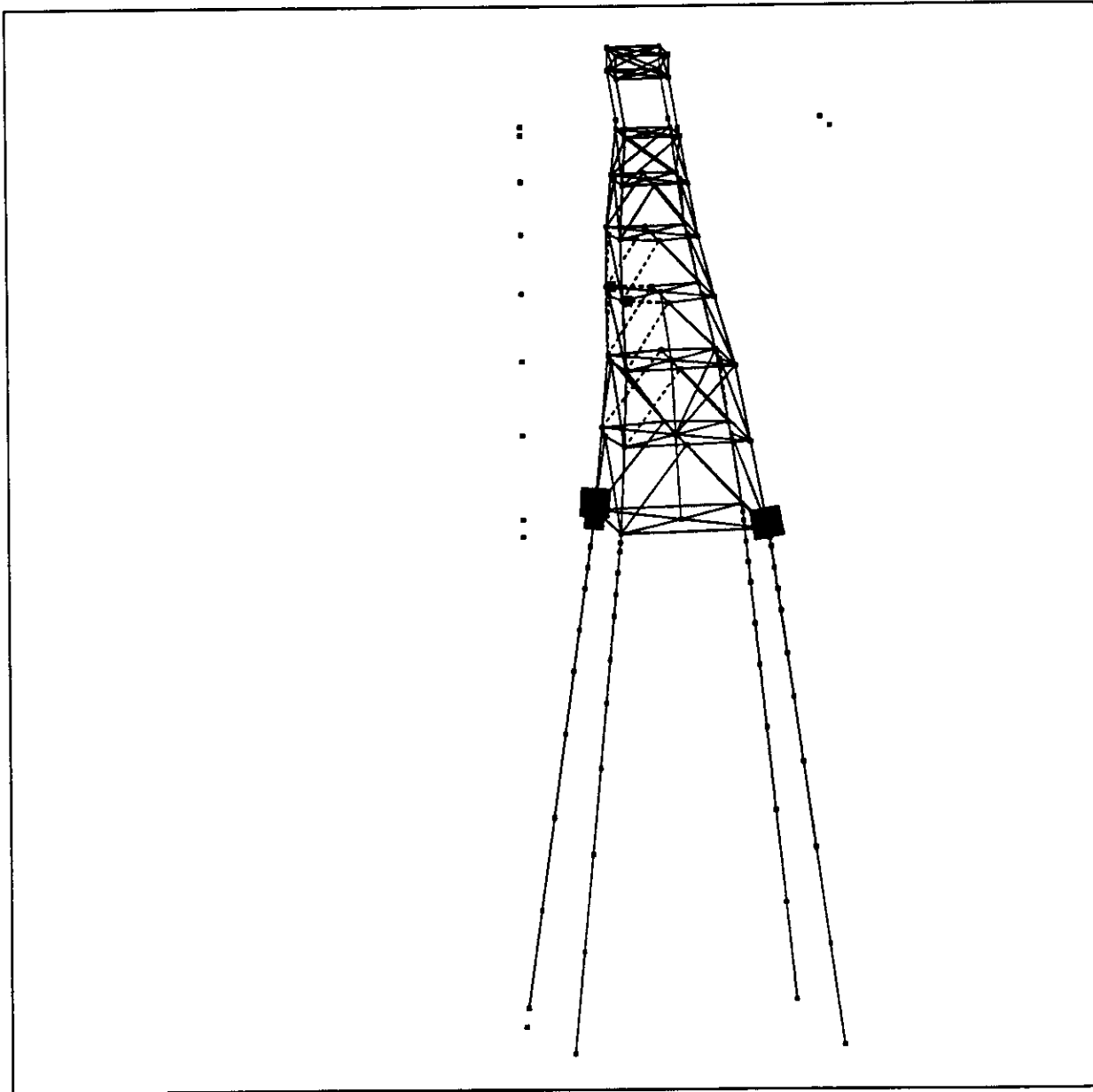
Soil Nodes Fixed Snapshot 1

Inelastic Events Legend

— Elastic  
 - - - - - Strut Residual  
 . . . . . Plastic Strut/NLTruss  
 = Beam Clmn Fully Plastic

- - - - - Strut Buckling  
 = Strut Reloading  
 . . . . . Beam Clmn Initial Yield

**Figure 6-12 Fixed Base: Load Step at First Yield**

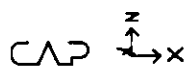
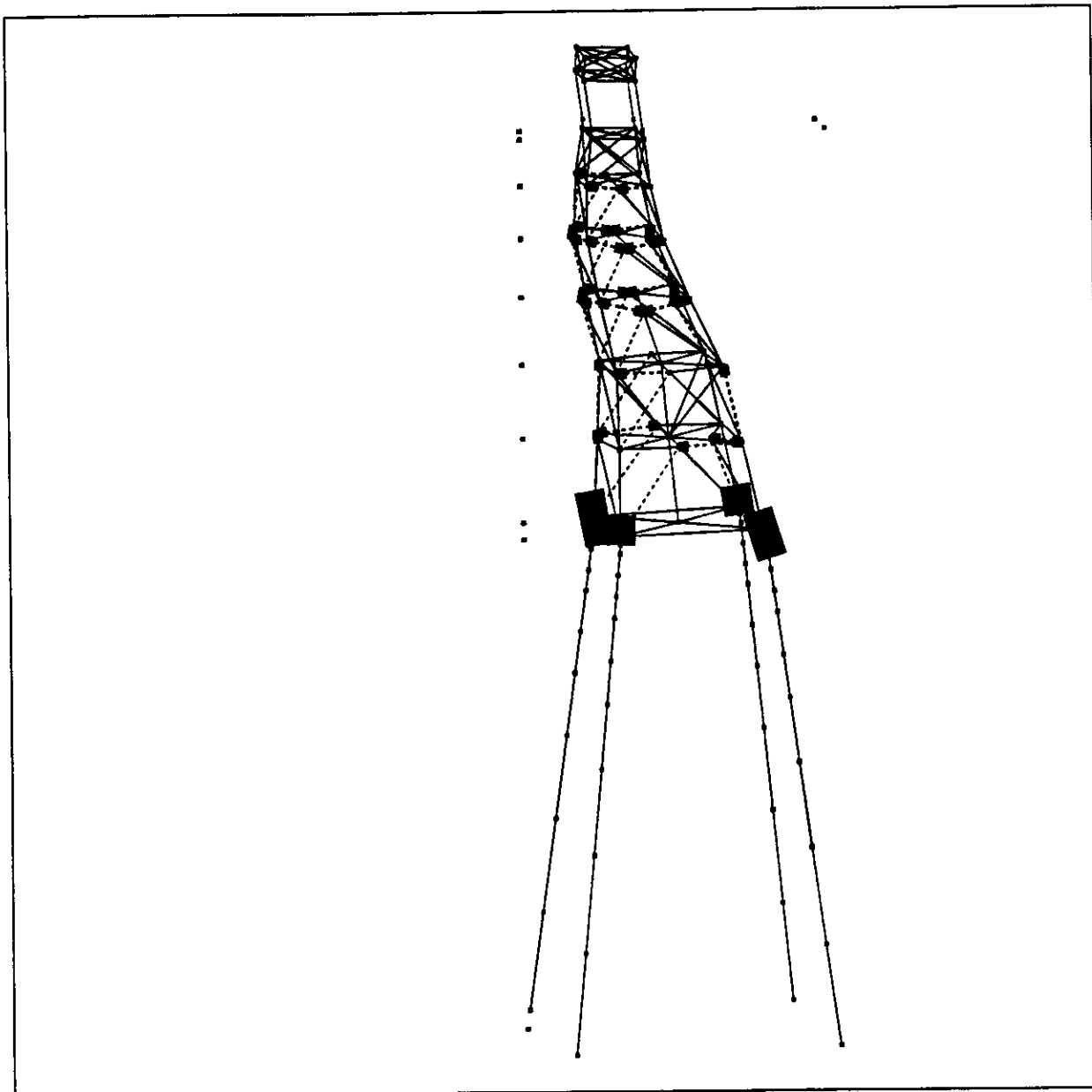


Soil Nodes Fixed Snapshot 2

Inelastic Events Legend

————	Elastic	-----	Strut Buckling
-----	Strut Residual	————	Strut Reloading
.....	Plastic Strut/NLTruss	.....	Beam Clmn Initial Yield
————	Beam Clmn Fully Plastic		

**Figure 6-13 Fixed Base: Intermediate Load Step**



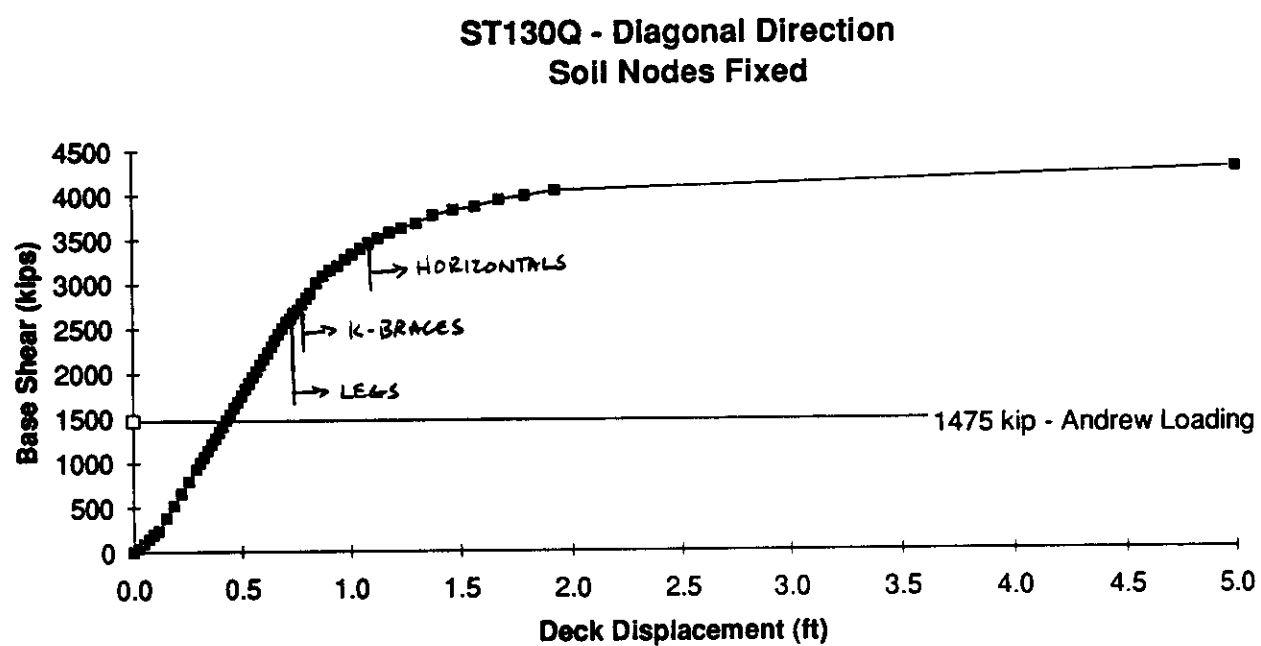
Soil Nodes Fixed Snapshot 3

Inelastic Events Legend

\_\_\_\_\_ Elastic  
 ..... Strut Residual  
 ..... Plastic Strut/NLTruss  
 \_\_\_\_\_ Beam Clmn Fully Plastic

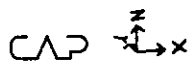
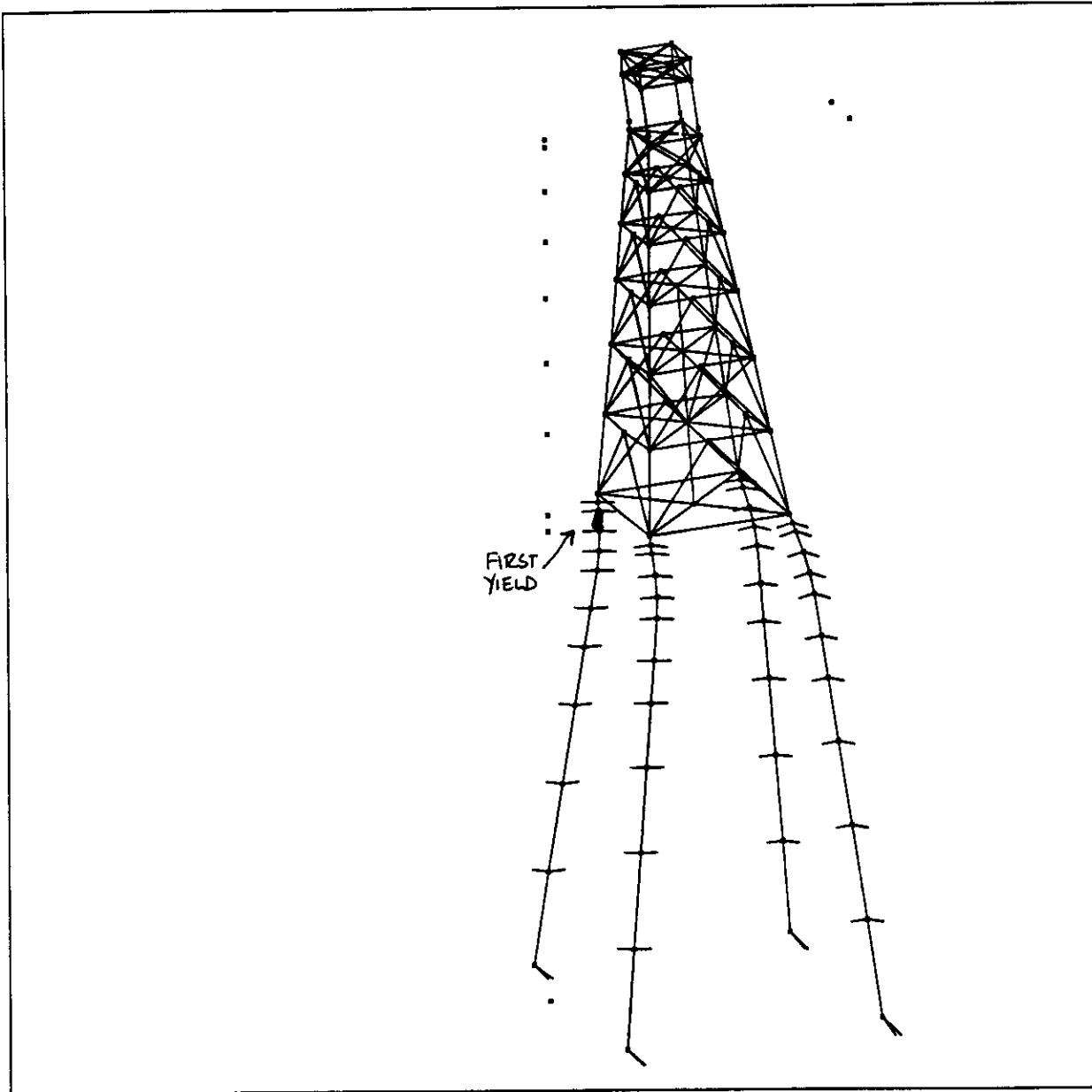
..... Strut Buckling  
 \_\_\_\_\_ Strut Reloading  
 ..... Beam Clmn Initial Yield

**Figure 6-14 Fixed Base: Last Load Step**



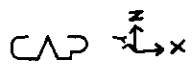
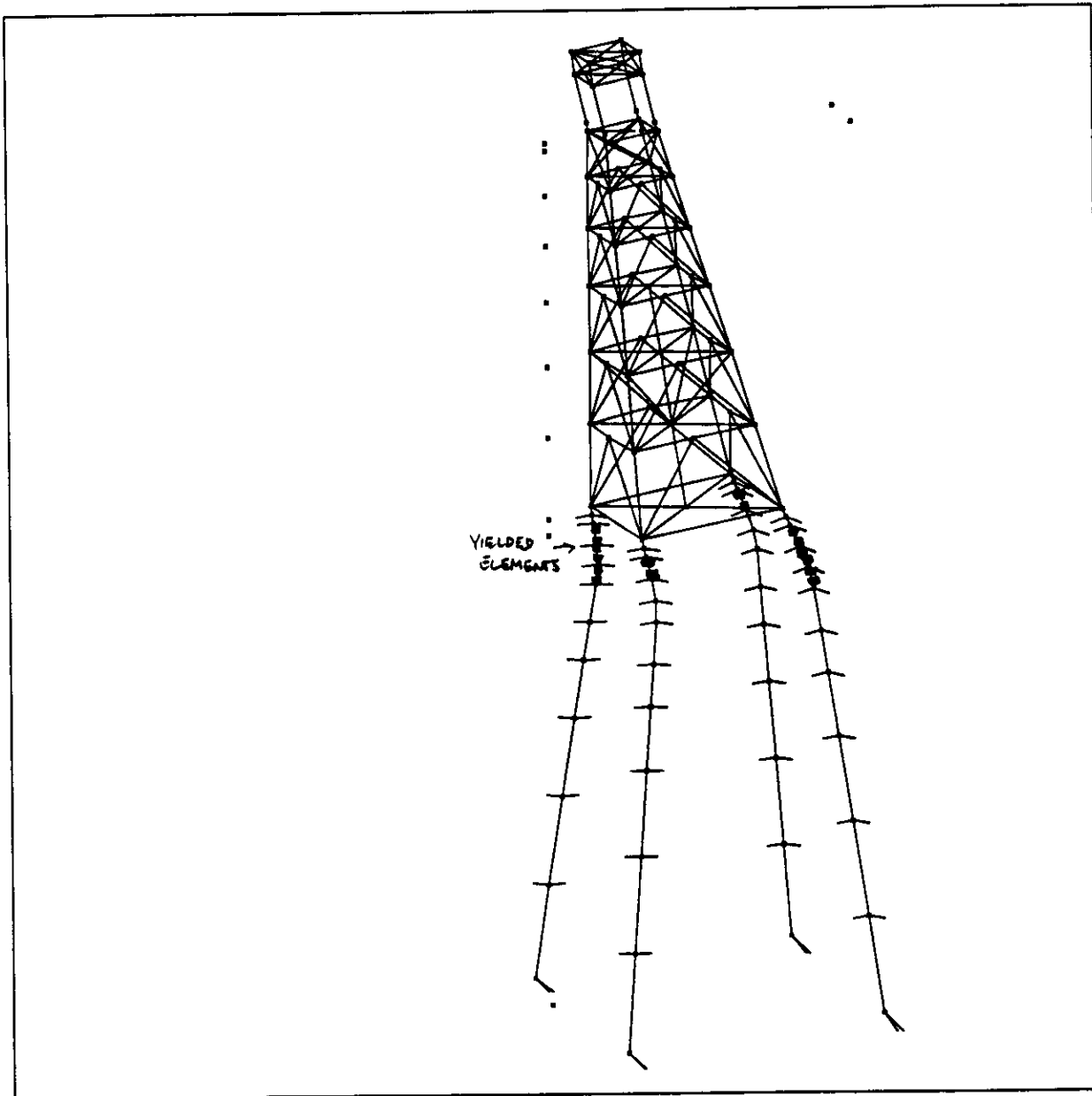
**Figure 6-15 Fixed Base Results**





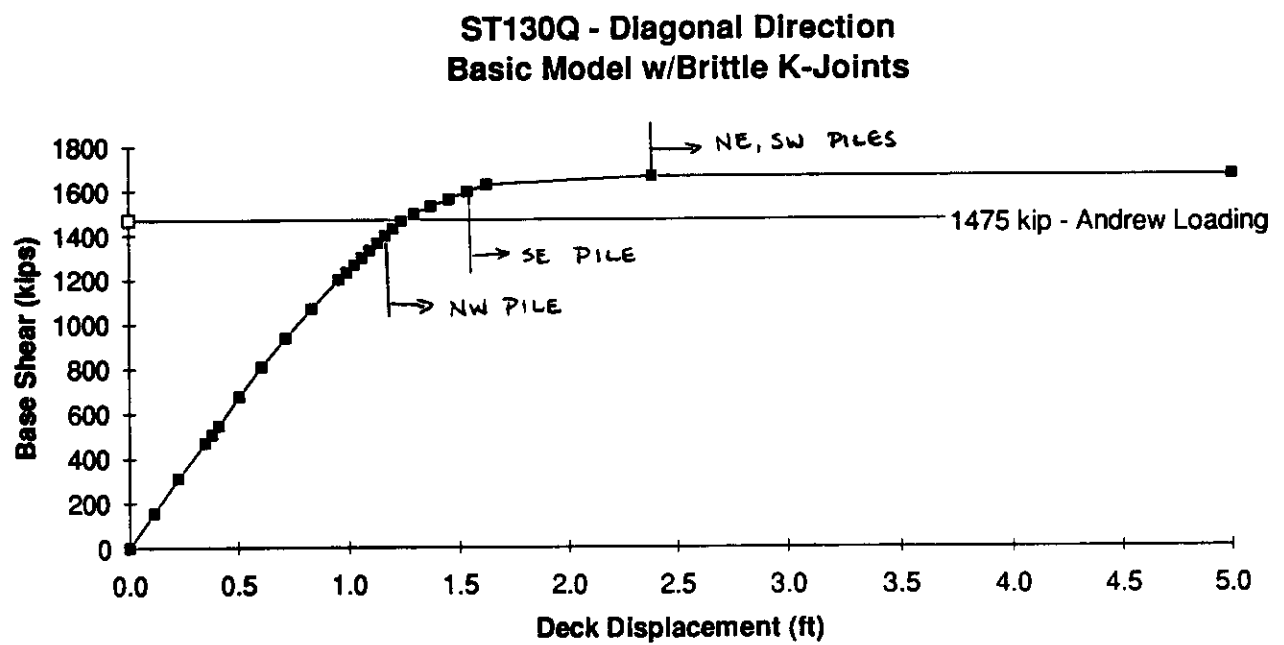
Basic Model w/Brittle K-Joints Snapshot 1

**Figure 6-16 Brittle Joints: Load Step at First Yield**

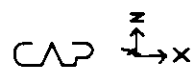
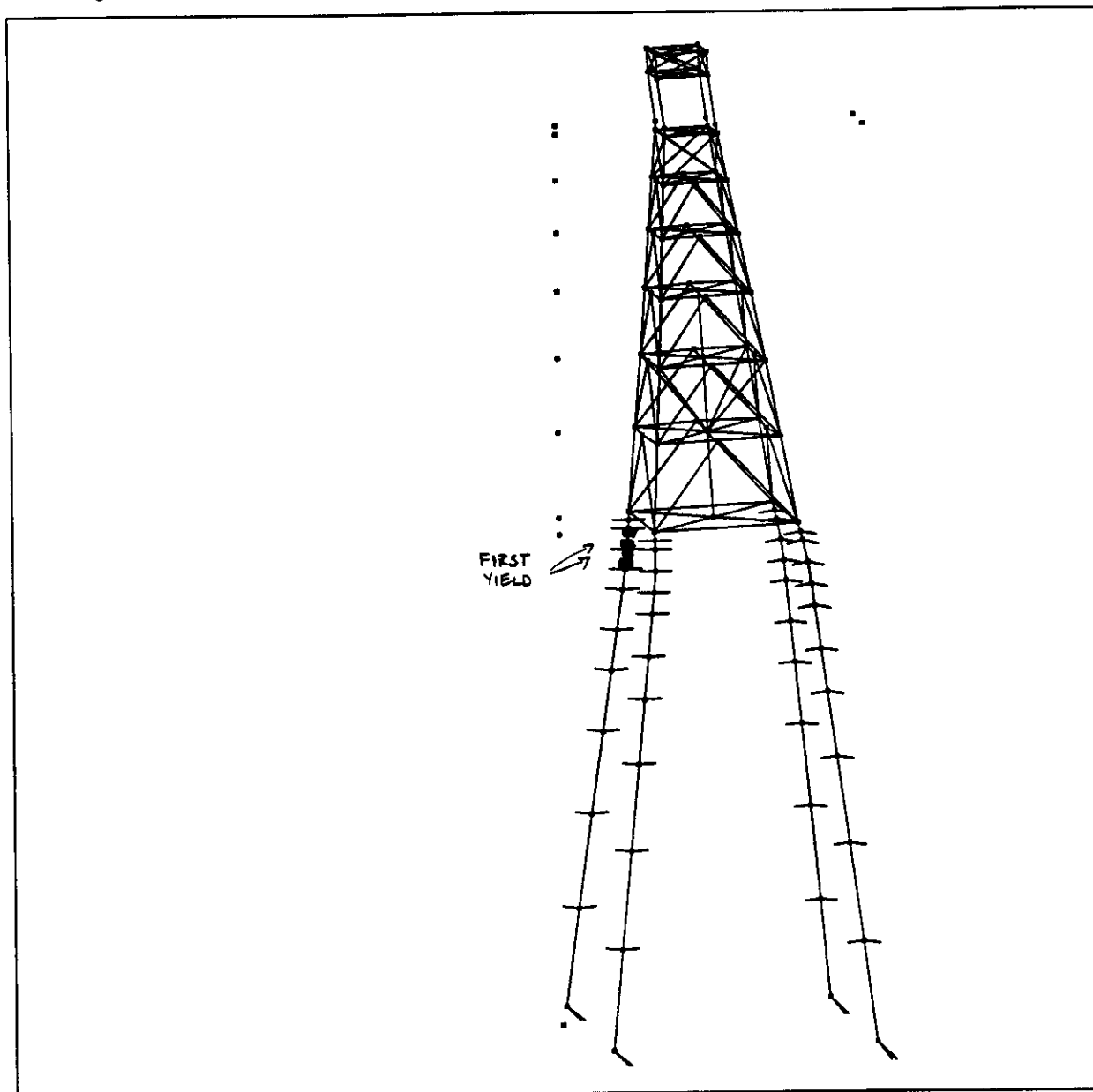


Basic Model w/Brittle K-Joints Snapshot 2

**Figure 6-17 Brittle Joints: Last Load Step**

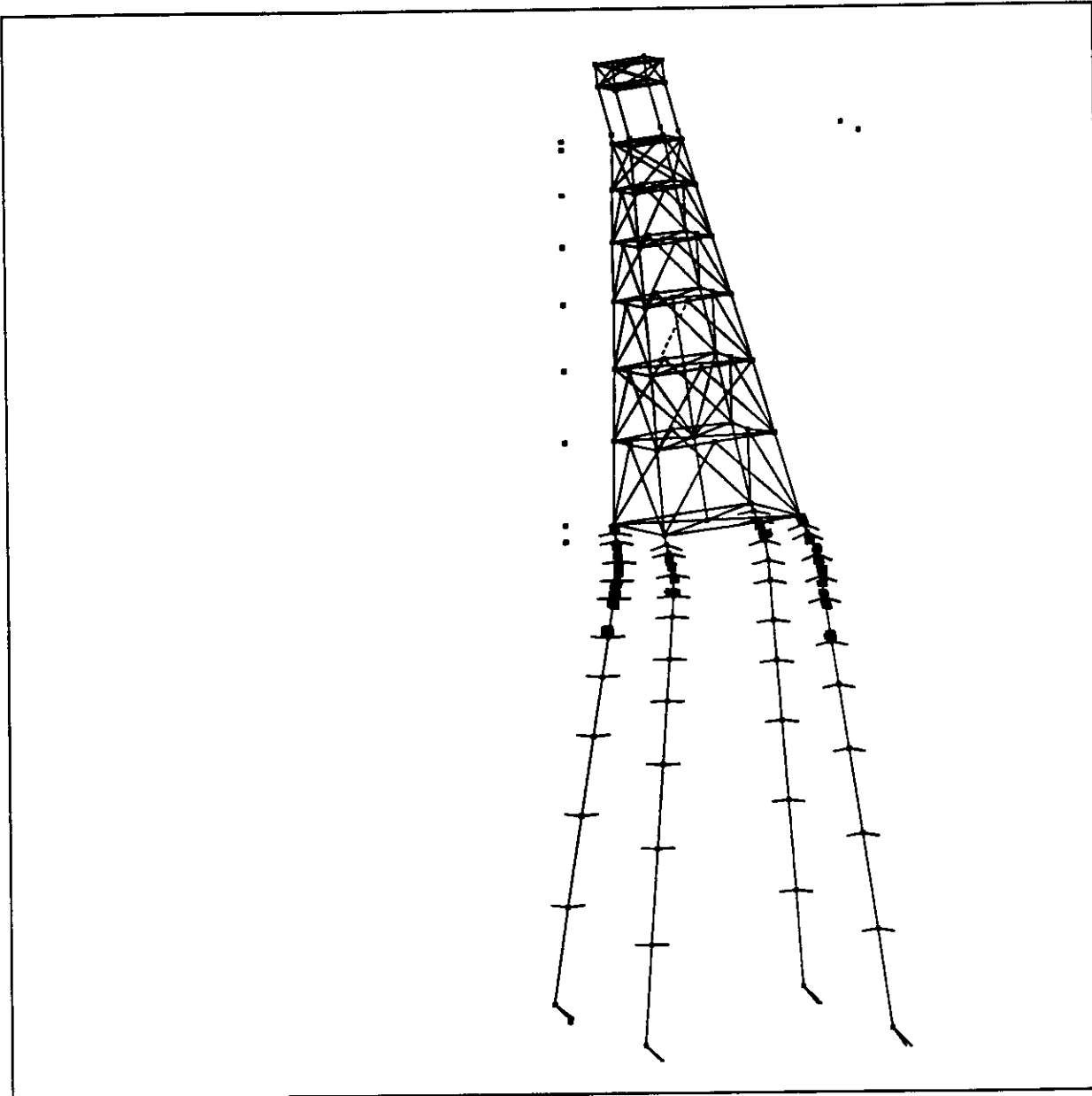


**Figure 6-18 Brittle Joints: Results**



Soil Imp. 200% w/Brittle K-Joints Snapshot 1

**Figure 6-19 Brittle Joints (200% Soils): Load Step at First Yield**



CAP  $\uparrow$  x

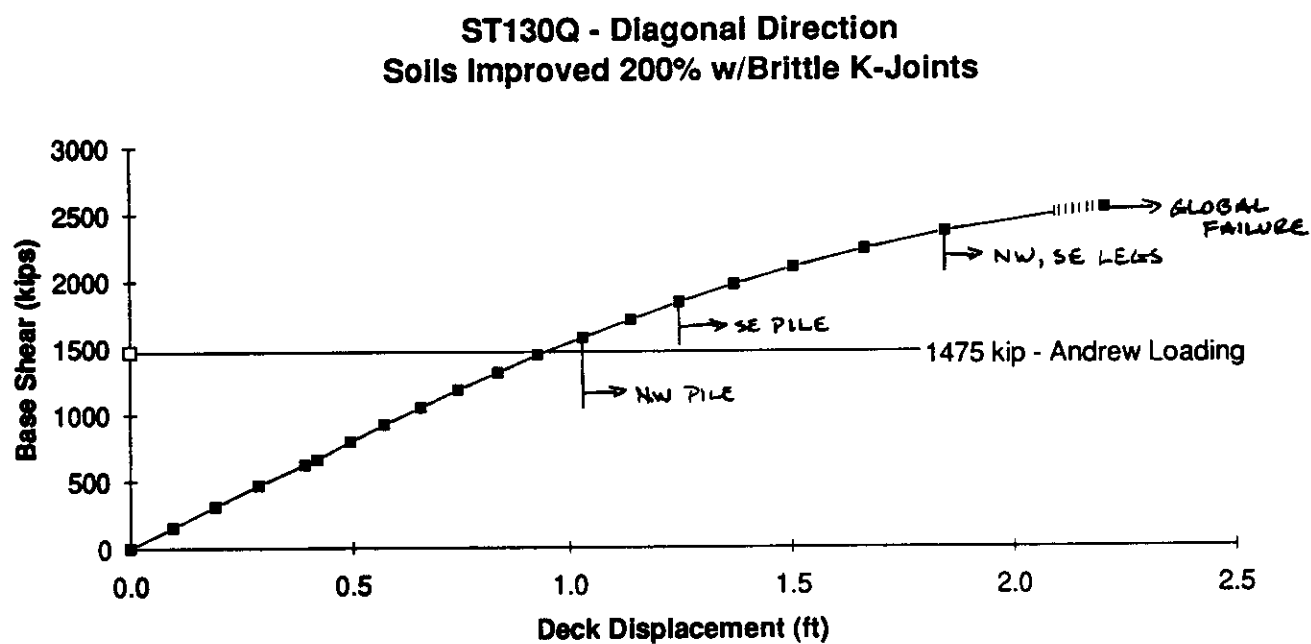
Soils Improved 200%, Brittle K-Joint Snapshot 2

Inelastic Events Legend

\_\_\_\_\_ Elastic  
 - - - - - Strut Residual  
 . . . . . Plastic Strut/NLTruss  
 \_\_\_\_\_ Beam Clmn Fully Plastic

- - - - - Strut Buckling  
 \_\_\_\_\_ Strut Reloading  
 . . . . . Beam Clmn Initial Yield

**Figure 6-20 Brittle Joints: (200% Soils): Last Load Step**



**Figure 6-21 Brittle Joints (200% Soils) Results**

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## SECTION 7 CONCLUSIONS AND RECOMMENDATIONS

### 7.1 Platform Inspections

The detailed platform inspections performed by this project indicated no significant findings above and beyond that originally found by the platform operator during the visual inspections required by MMS NTL 92-07 immediately following Andrew. Although only three platforms were inspected, this is probably a good indication that post storm visual inspections provide a reasonable means of determining platform integrity following a major storm such as Andrew. The visual inspections can be performed in a brief period of time (several hours vs. 1-2 days for detailed inspection) allowing a large number of platforms to be evaluated in a short period of time, thereby providing a quick, yet accurate picture of the offshore platform fleet and which platforms need immediate attention.

A detailed inspection of the region of the ST 151 K platform that was found to have "popped" marine growth during the operator's post Andre visual inspection indicated no damage. This region was specified for inspection based upon similar "popped" marine growth found by Amoco (Imm, et. al. 1994) which indicated some member yielding. For the ST 151 K platform, this "popped" marine growth does not appear to be from member yielding and is instead likely from a falling object, perhaps when the wave impacted the platform's deck forcing debris overboard. The conclusion is that this type of visual damage does not always indicate a structural problem.

The only two "defects" found in this inspection but not the earlier visual inspections do not appear to impact platform integrity. The missing weld material on ST 151 K (Figure 2-5 and Photo 2-16) has apparently been there since the platform was installed over 30 years ago and there is no indication of crack growth or any other detrimental effects. The missing weld is only about 1 inch long and is along a generally speaking, non critical horizontal member (a vertical diagonal would perhaps be a different story) and therefore should not impact structural global performance. The crack found on ST 130 Q is of greater concern since it is on a vertical diagonal that is heavily loaded during large storms. However, the member is not flooded in the region of the crack indicating that it is not a through crack and may instead be a "laminar" crack. Similar to the missing weld material on ST 151 K, this crack may also have existed for many years. Since the crack is not through and since it is perpendicular to the joint of this primarily axially loaded member, it is not anticipated to significantly impact platform integrity. However, the operator may want to investigate further and perhaps monitor the crack to ensure that it does not develop into a larger problem.

Ideally, the study would have inspected several more platforms, particularly platforms of other operators. This would provide more insight to inspection of different platform types operated by different organizations using different inspection strategies. However, this was not possible



with the time frame and budget available for this study. In addition, an inspection of this type requires significant cooperation from the platform operator which is not always forthcoming. This study was fortunate to have considerable cooperation from Chevron.

As noted below, several of the structural analysis indicate that some form of foundation damage (pile bending) should have been noted on the platforms. Yet none was observed. However, it is difficult to "observe" a foundation failure due to inadequate visibility at the mudline and lack of a frame of reference (is the base level?) and the obvious fact that the piles are below the mudline with no chance for direct inspection. Thus some of these (and other platforms in Andrew may have sustained foundation damage but the damage is not observable. Another possibility is that the analytical models used to predict foundation behavior (e.g. API RP 2A) are conservative. This point is currently being studied in a project funded in part by the MMS (PMB, 1994).

### **7.2 Platform Analysis**

The structural analysis used results of the Andrew JIP to improve and update computer models for use in this project. The improvements were related to the joints and the foundation. In addition, the models were updated to account for such things as number of boat landings and barge bumpers actually observed on the platform during the inspections.

The joints were updated on two fronts - capacities and force-deformation characteristics. Capacities used the latest information available in the public domain. The resulting joint capacities were higher than that used in the Andrew JIP which were based upon the joint capacity equations in API RP 2A (factors of safety excluded). For force deformation characteristics, the joints were modeled as elastic-plastic (similar to that used in the Andrew JIP) and elastic-brittle. The elastic brittle appears to more realistically reflect actual performance which was seen to be complete shear at the joint as evidenced in the post Andre inspection of the ST 177 B platform (PMB, 1994d). The foundation modeling improvement consisted of a 100 percent increase in the soil strength above that determined using API RP 2A recommendations. This increase forces the failure mode up into the jacket, which matches more closely with the observed field behavior.

The results of this revised modeling indicate better matching of the analytical to actual platform performance. The Andrew JIP generally found that all of the structures were "unexpected" survivals in Andrew. This project however, with the revised joint and foundation modelling, does not indicate such clear unexpected survivals. All three platforms appear to fail at about the level of the Andrew loading, whereas in the Andrew JIP, the platforms appeared to fail at a loading clearly below that of Andrew. However, only one of the platforms actually failed in Andrew.

Results for the toppled ST 130 A platform provided the most information and it was found that the analysis could be "tuned" so that the analysis results match observed results. In this case it was determined that the computer model with brittle joints, a foundation modeled using increased (100%) soil strength, and a localized ungrouted pile-leg annulus (reflecting a poor grout job during the original platform installation) predicts a platform failure similar to that observed during Andrew.

Results for the ST 130A and ST 130 Q platform were less informative, partly because the platforms exhibited no damage that could be compared to that predicted by the computer model. The joint and foundation modeling improvements did increase the level of loading required to fail the platform; however, the computer models still predicted substantial damage to the platform's joints and foundation, yet no storm related damage was observed during the inspections.

As previously noted, the computer models of this study would also label these platforms as unexpected survivals. One possible explanation is wave loads on the deck. The analytical model assumes full impact of the wave across the entire deck resulting in large deck loads. However, discussions with Chevron indicate that based upon observed damage to the deck, waves impacted the deck, but only in certain areas, and most likely not across the entire deck width in a "long crested" fashion. If this was accounted for in the computer model, the total loads acting on the platform would reduce significantly and perhaps the computer model would not predict the large extent of damage to the joints and foundation.

At this time it appears that computer models similar to those used in this project can be used to identify platforms that may be at risk in large hurricanes. This study confirms that the ultimate capacity approach recommended in API RP 2A Draft Section 17 provides a reasonable estimate of platform performance and is appropriate for use in assessment of existing platforms.

This study, as well as the Andrew JIP, have shown that if properly performed, these types of analysis are generally conservative. However, additional work is recommended, and in some cases is ongoing, to further refine these types of analysis so that predictions of platform performance can be more accurate. Recommendations include:

- Joints. The most significant variation appears to be joint capacity and joint modeling. Further studies are needed that address both joint capacity and modeling such as the proposed Andrew JIP Phase II (PMB, 1994b) and the three dimensional Frames Test in the UK by Bomel.

## MMS Platform Inspections and Analysis

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- Foundations. There appears to be conservatism in the API RP 2A formulations typical used by operators to estimate foundations strength. The ongoing API/MMS study of foundation behavior during Andrew should help address this issue particularly since some of the study focuses upon observed foundation failures of caissons. Further observation and associated analytical evaluation of a platform that experienced a foundation failure would be most helpful.

- Wave-in-Deck. For the wave-in-deck condition, it appears that current approaches to estimate the deck loads may be too conservative, since the wave does not always impact the full length of the deck in a "long crested" fashion as assumed in the analytical models. Perhaps a reduction factor similar to the spreading factor of 0.88 that is used for calculating loads on the jacket would be appropriate. However, for platform assessment-type work, it is conservative to make the "long crested" assumption and is the best approach available until further field information and observations or model testing of waves impacting decks is available.

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MMS Platform Inspections and Analysis

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**APPENDIX A      INSPECTION REPORT**

**(See Separate Document Including Video)**